VEHICLE WEIGHTS AND DIMENSIONS STUDY

TRAFFIC STREAM EFFECTS
OF
PROPOSED TRUCK LENGTH INCREASE

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<u>Prepared by</u>

John B.L. Robinson
Special Advisor
VW&D Study

ABSTRACT

This report provides a review of the effects on the traffic stream of an increase in the allowable maximum overall length of heavy trucks from 23 meters to 25 meters. Five major issues were identified as being of concern; stability and control performance, aerodynamic effects, signing practice, capacity and level of service issues and passing operations. With the exception of the stability and control issues, which were examined in detail in the course of the Vehicle Weights and Dimensions Research Program, these areas are discussed in this report.

The work was based on a review of the current published literature in this domain, and the issue of greatest significance was found to be the operation of vehicles of 25 meters on two-lane, two-way (TLTW) highways. The report concentrates on this problem area, setting out a clear distinction between DESIGN passing sight distance requirements and BARRIER LINE sight distance requirements. The report concludes that current Canadian barrier line practice as set out in the Manual of Uniform Traffic Control Devices for Canada (MUTCDC) is significantly higher than both current U.S. and Australian standards. Although a clear need for research in this area was identified, currently available information would seem to indicate that an increase to 25 meters in the allowable maximum overall length for heavy vehicles operating under standard regulations could be tolerated at this time.

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NOTE

ALL FIGURES, TABLES AND APPENDICES WILL BE FOUND AT THE END OF THE REPORT IMMEDIATELY FOLLOWING THE REFERENCES.

BIBLIOGRAPHY

Note

An extensive bibliography of relevant articles has been compiled through a search of the TRIS and IRRD data bases. It contains about 100 references and is too extensive to include in this document. Anyone interested in obtaining a copy should contact the RTAC Secretariat.

1. INTRODUCTION

1.1 Background

The need to review the effect of longer heavy vehicles on other elements of the traffic stream under various operating conditions arose form a general consideration of increasing the maximum allowable overall length of vehicles from 23 meters to 30 meters. After initial work and discussion of such a consideration a more modest scenario which would see an increase from 23 meters to 25 meters was agreed upon.

This report discusses a number of outstanding questions on traffic stream effects in the context of this latter scenario. It is important to note that parallel recommendations being put forward regarding the maximum allowable gross combination weights (GCW) on heavy vehicles would see no increase above the levels currently allowed in certain parts of the country. In fact, for certain types of vehicles, maximum allowable GCW's would decrease.

1.2 Report Focus

This report focuses its discussions in a directed context as defined by two main constraints. First, the report addresses only the LENGTH issue. As noted above, there is no increase in GCW proposed in the work stemming from the Vehicle Weights and Dimensions (VW&D) Study. The issue of the effects of increased weight thus does not exist and is not considered.

Secondly, the work deals only with an incremental increase in maximum allowable overall length from 23 meters to 25 meters, roughly a 9% increase. The 23 meter maximum is currently in force in eight of twelve jurisdictions in Canada and fundamental to this analysis is the assumption that it is not causing any undue safety related problems at this time.

The remainder of this report discusses the traffic stream effects of the proposed length increase under four main headings:

* Aerodynamic effects
    - buffeting
    - splash and spray

* Signing Practice
    - sign shadowing

* Capacity and Level of Service Issues
    - General Effects
        - operating speeds
        - acceleration characteristics
    - Specific Issues
        - urban intersections
        - storage lanes
        - section capacities
        - weaving sections
        - merge/diverge areas

* Passing Operations
    - multiple lane facilities
    - two-lane, two-way facilities

It was also recognized that issues in the area of vehicle stability and control performance arose in considering possible increases in maximum allowable length. Four elements of stability and control were identified:

- braking characteristics
- stability performance
- offtracking characteristics
- swing-out

These issues have been treated rigorously in the part of the Vehicle Weights and Dimensions (VW&D) research program dealing with vehicle stability and control (1). Technical principles deriving from consideration of these issues have been incorporated into the proposals currently being put forward and will not be covered further here.

2. AERODYNAMIC EFFECTS

2.1 BUFFETING

The aerodynamic buffeting experienced by smaller vehicles passing heavy trucks is a well known phenomenon. The question of aerodynamic effects was discussed in a review of the possible impacts of longer combination vehicles carried out by the United States Transportation Research Board in 1986 (2). Increases in overall vehicle length will have little or no effect on the force to which the passing vehicle is subjected. While exposure times to such buffeting do obviously increase with length, at 100 kmh the increase would amount to less than a tenth of a second for for the change under consideration.

The question of the effect of a second gap in double combination vehicles is unimportant. There is no difference between passing a double combination vehicle of given length versus passing a tractor/semi-trailer of the same length from the standpoint of aerodynamics.

2.2 Splash and Spray

The splash and spray question is one which relates to aerodynamics, but which is poorly understood at the present time. Research cited in the TRB report noted above (3) suggests that there is no conclusive evidence that longer combination vehicles would be better or worse from this point of view. Obviously, if trucks get longer, the time for which the passing vehicle is subjected to splash and spray in wet weather increases. As noted above however, the exposure time increase for a 2 meter change in length is almost negligible. Even if one considers that in the wet the passing manoeuvre is executed at lower speeds such as 80 kmh, the additional time required to pass a 25 meter over a 23 meter vehicle would only be slightly more than a tenth of a second.

2.3 Conclusions: Aerodynamic Effects

Changes in overall length from 23m to 25m would appear to have little or no aerodynamic impact on other vehicles in the traffic stream, whether it be from a buffeting or a splash and spray standpoint.

3.Ø SIGNING PRACTICE

3.1 Sign Shadowing

A recent paper published in the Research Record of the U.S, Transportation Research Board (4) brought into question the effects of longer vehicles on current sign placement practice in the U.S. The concern expressed in the paper dealt with the problem of "shadowing" of signs by longer vehicles which would effectively impede perception of those signs by the occupants of smaller vehicles.

While the potential for a problem would seem to exist, the report points out that there is no definitive research work in this regard and that accident information which might confirm its presence does not exist. In addition, it is recognized that motorists placed in such a situation generally take some countervailing action to overcome the problem. The fact that signing is also usually placed in a duplicative manner (ie: advanced signing plus "on-site") further reduces the probability that such shadowing problems might cause any serious disruption of traffic flow or safety related problems.

3.2 Conclusions: Signing Practice

Recognizing the minor nature of the length increase being considered and the discussions noted above, it would seem that signing considerations do not offer any real impediment to increasing maximum allowable overall vehicle lengths to 25 m.

4.0 CAPACITY AND LEVEL OF SERVICE ISSUES

4.1 General Considerations

4.1.1 Operating Speeds

Changes in operating speeds of heavy combination vehicles are related to increases in gross combination weight (GCW) allowances. Increases in GCW without concomitant upward changes in motive power will necessarily degrade both the acceleration characteristics and the operating speeds of heavy trucks on grades.

The effects of increases in GCW on the operating speeds of trucks, and consequently on the capacity and levels of service on different types of facility have been well documented in the literature. A particularly good summary will be found in the Transportation Research Board Special Report 211 (5).

The changes in overall length allowances being considered in this report are not being accompanied by increases in allowable GCW however. As was previously pointed out, some types of truck combinations are in fact being allowed lower GCW's under the current proposals. Thus, operating speeds of heavy vehicles that do make use of the additional length allowance can be expected to remain within the performance envelope of those currently on the road.

4.1.2 Acceleration Characteristics

It is recognized that the acceleration characteristics of heavy trucks are very different from those of cars. As with operating speeds the weight to power ratio of the vehicle is the critical determinant from an acceleration performance standpoint. Decreases in accelerative performance can negatively effect levels of service and capacity in a number of situations on two-lane roads, multi-lane facilities and at intersections. Once again, these effects are well documented in the literature, with a good summary being provided in the TRB report cited previously (6).

As noted in the case of operating speeds however, GCW increases are not being proposed in the current scenario. The proposed length increase will thus not be accompanied by a degradation of the accelerative performance of heavy vehicles currently using the system.

4.2 Particular Issues

4.2.1 Urban Intersection Capacity

Concern for the effect of an increase in the overall length of trucks on intersection capacities has been expressed. The presence of trucks in a traffic stream is generally recognized through the use of passenger-car equivalents (PCE's) (7),(8). While work by Keller and Saklas (9) suggests that PCE's used in the U.S. Highway Capacity Manual may be too high, nonetheless such factors applied to observed traffic volumes and classification breakdowns allow the traffic engineer to recognize the longer time required for heavy vehicles to execute intersection manoevres due to their inferior acceleration characteristics. Length is not a variable in the determination of PCE's. In some methods power to weight ratios or their inverse are used in recognition of the dominant role of performance in determining the vehicle's impact on surrounding traffic. From this standpoint then the proposed length increase should have no effect.

While any increase in overall length by 2 meters may in theory change the requirement for intergreen times slightly, the marginal difference this would make would be meaningless in the context of the probabilistic approach currently used in intersection analysis. In fact, the level of service offered at an intersection will be much more sensitive to the percentage of longer vehicles in the traffic stream than to any marginal change in length of the kind proposed.

4.2.2 Storage Lane Length Requirements

Storage lane length requirements for turning vehicles at intersections are generally predicated on a number of factors including:

- an "average vehicle length"
- the probability of the vehicle clearing the intersection on the first green
- the vehicle arrival rate
- the vehicle arrival distribution

The final requirement for storage lane lengths is far more sensitive to the latter three variables than to the first.

Average vehicle lengths can be calculated quite simply knowing the percentage of different types of vehicles in a traffic stream and using an average length for each type. Trucks generally make up only a small minority of the traffic stream. Heavy articulated trucks in turn are often only a minority of the truck population in most parts of an urban area. Further, because of other dimensional constraints put forward in the proposals under consideration, only a small percentage of the heavy truck fleet will be able to take advantage of any increase in length over 23 meters. It is thus anticipated that the proposed length increase will have a negligible effect on storage lane length requirements at urban intersections.

4.2.3 Section Capacities

Three different road section situations were reviewed:

- section capacities
- weaving section capacities
- merge/diverge area capacities

Short of very sophisticated simulation techniques, the Passenger Car Equivalents (PCE) concept used in intersection capacity analysis is also applied in accounting for the presence of trucks in the traffic stream when analyzing capacities and levels of service for the types of road sections listed above.

Once again, the PCE's used for such analyses do <u>not</u> depend on length as a critical variable but rather depend on the power to weight ratio of the vehicle. With commonly used methodologies such as the U.S. Highway Capacity Manual (10) a proposed change of 2 meters in overall length would have no effect on the calculated level of service or capacity of any of the section types noted above. The critical factor in all of these sections is the presence of grades and when this occurs, it is changes in the power to weight ratio that might effect PCE's, not changes in length. Since no GCW increases are being proposed, this issue is not of concern in the scenario being analyzed.

It is worth noting that work by Craus, Polus and Grinberg (11) suggests that in fact PCE's used for section capacities in the Highway Capacity Manual may be too high and thus may be giving somewhat conservative estimates of capacity and levels of service on these road sections.

4.3 Conclusions: Capacity and Level of Service Issues

In summary it appears that the proposed increase in maximum allowable overall vehicle length will not change in any significant way the impact of the current truck fleet on capacity and level of service on the various elements of the roadway system discussed above. This is attributable primarily to the fact that the traffic stream impacts of heavy vehicles on capacity and level of service are mainly due to their particular operating speed and accelerative characteristics. These in turn depend on the power to weight ratios of such vehicles and are not generally a function of overall length. Since the proposed two meter increase in length is not being accompanied by any increase in GCW, it follows that average fleet performance characteristics will not be effected by the increase in length.

5.0 PASSING OPERATIONS

5.1 Passing Operations: Multi Lane Highways

Passing operations problems are generally linked to the adequacy of passing sight distance provisions on a given facility. On multi-lane facilities operating under free flow or steady state conditions the provision of adequate passing sight distance is not a problem. This is the case simply because vehicles wishing to pass others on multi-lane roads may do so without using the opposing direction's lanes. Thus, the conflict with opposing traffic is eliminated in this case. Under forced or unstable flow conditions the question of passing opportunities on multi-lane facilities becomes irrelevant because of the highly constrained operating conditions.

Grades on multi-lane facilities influence the performance of heavy trucks and presumably generate increased demand for passing by other vehicles. However, as noted previously, the consequent impact on other elements of the traffic stream stems from power to weight ratio problems rather than any problems related to the length of the vehicle.

5.2 Passing Operations: Two-lane, Two-way Highways

In the course of preparing this report it became evident that the major area of concern regarding the impacts of the proposed 2 meter overall length increase would be in respect of passing operations on two-lane, two-way rural highways. In this situation, the passing vehicle uses the traffic lane of the opposing direction to carry out the manoeuvre. The time and sight distance required for the completion of this operation are critical to its safety and it is readily evident that even the proposed minor increase in length is going to have some impact, however small, on these parameters.

The passing manoeuvre however is a complex one and in addition to the physical parameters involved, it also depends to a major degree on both probabilistic notions and psychological factors. Current standards for various design and operational features of the road effecting passing operations appear to be based largely on empirical data which implicitly recognize some of these non-quantifiable aspects of the problem. More recent work using both simulation and kinematic modeling techniques has shed some new light on this area, but the current state of understanding is still to some degree in its infancy.

Thus, to a certain extent judgment is required in assessing whether or not the proposed two meter increase can be accommodated within current standards. To help provide the reader with a better perspective on the question and the final recommendation of this report a brief background discussion is provided below.

5.2.1 Elements of the Passing Manoeuvre

The analysis of passing manoeuvres on two-lane, two-way (TLTW) facilities requires a clear understanding of the nature of the manoeuvre involved and the consequent visibility requirements for the driver. While different interpretations have been put on the manoeuvre, the most generally postulated model of its constituent elements is illustrated in Figure 1.

These distance elements include:

* d1 REACTION TIME - the time for the driver to make the decision to pass

* d2 LEFT LANE OCCUPANCY DISTANCE - the distance during which the passing vehicle occupies the opposing lane

* d3 CLEARANCE DISTANCE - the distance between the passing vehicle and an opposing vehicle at the completion of the manoeuvre

* d4 OPPOSING VEHICLE TRAVEL DISTANCE - the distance travelled by the opposing vehicle during the time it takes the passing vehicle to cover the pass completion distance

Central to this description of the passing manoeuvre is the concept of the CRITICAL POSITION of the passing vehicle. The critical position as defined by Weaver and Woods (12) occurs:

> "... when the passing vehicle in the left lane is abreast of the vehicle in the right lane being passed. When the passing vehicle reaches this point, the commitment to complete the pass is reached; it is safer to complete the manoeuvre than to abort and attempt to pull back in behind the vehicle being passed."

A safe passing manoeuvre thus requires two things:

* That the pass can be successfully <u>aborted</u> if an opposing vehicle comes into view while the passing vehicle is in the left lane <u>before</u> reaching the critical position.

* That the pass can be successfully completed should an opposing vehicle appear while the passing vehicle is in the left lane at or after reaching the critical position.

The sight distance required from an operational standpoint thus should be measured from the critical position rather than from the beginning to the end of the pass. This concept leads to an important distinction between "design-based" passing sight distance and the operational considerations developed from the passing manoeuvre as outlined above. Weaver and Woods (13) define this distinction between passing sight distance for design and the operational requirement which they term passing sight distance for barrier lines.

> "The design philosophy is that if total sight distance throughout the entire manoeuvre is provided at the beginning of a passing opportunity, a driver may sequentially execute each element comfortably with full visual knowledge throughout the complete manoeuvre of the presence of an opposing vehicle that may be within the required passing distance and allow adequate separation distance between the two opposing vehicles at the completion of the manoeuvre. Provision of sight distance to permit this type of operation represents a desirable design objective. Provision of less available sight distance than this will necessarily produce less than desirable design. The amount of available sight distance can be reduced below this total distance value until a point at which sight distance becomes less than that necessary to perceive an opposing vehicle in time to safely complete a passing manoeuvre once the driver is committed to the execution of the manoeuvre. This in essence, predicates `minimum passing sight distance' and forms the basis of the marking sight distance definition."

The distinction made here is important for two reasons. First, it establishes a clear difference between design passing sight distance (PSD) and barrier line PSD. PSD for design cannot be used for marking barrier lines and PSD for barrier lines cannot be used for design. Second, it is evident that from an operational standpoint, (ie. when a passing manoeuvre is actually being carried out), the critical sight distance is the barrier line PSD. The quantitative distinction between Design and Barrier Line PSD's is evident from Table 1.

It is interesting to note that if the Barrier Line PSD requirement is taken as previously defined - that is the provision of sufficient sight distance to allow a vehicle at the critical point to successfully complete the manoeuvre upon the appearance of an opposing vehicle in the left lane - then barrier line striping criteria as they now stand would not need to be changed. Such an approach, while in keeping with the current definitional practice, ignores the fact that the abort mode of a passing manoeuvre is directly effected by vehicle length. This question is discussed further in a following section.

5.2.2 Current Barrier Line Practice: Some Examples

Table 2 provides a comparison of a number of standards and recommendations for Barrier Line PSD practice from several sources for a range of speeds. Such a comparison helps put current Canadian practice into context when assessing whether or not the proposed two meter increase can be accommodated. A number of appendices are included with this report that provide the technical background to some of these standards including:

* APPENDIX A - extracts from pertinent sections of the MUTCDC on barrier line practice in Canada.

* APPENDIX B - extracts from the 1986 edition of the RTAC Geometric Design Manual regarding passing sight distance requirements.

* APPENDIX C - extract from "Basis of Standards" chapter of the 1986 edition of the RTAC Geometric Design Manual containing additional information on the technical basis for design PSD requirements.

* APPENDIX D - extract from the AASHTO "Green Book" (1984) discussing current AASHTO practice in design PSD requirements.

* APPENDIX E - extract from U.S. ITE Traffic Engineering Handbook discussing Barrier Line and Design PSD requirements.

In respect of Barrier Line PSD requirements, both U.S. and Australian standards currently lie substantially behind Canadian practice. Within Canada, Alberta has one standard which it applies to all roads requiring striping. The distance requirement is 425 m in recognition of the fact that the vast majority of their facilities requiring barrier line striping have 100 kmh speed limits. This distance is only slightly higher than the 400 m in the MUTCDC for the same speed.

Quebec has two standards, a desirable and a minimum. The minimum standard is in keeping with the MUTCDC however the desirable standard is significantly higher. What percentage of the network is marked at the desirable level versus the minimum level in Quebec is not known.

The technical basis for current Canadian standards of both Design and Barrier Line PSD is obscure. Apparently both are empirical in nature and the Design PSD would seem to stem from the same source as the AASHTO Green Book. Ironically, the latter is based on observations carried out initially in the late 30's and 40's, (see appendix D). While subsequent more recent studies were carried out it appears that no significant changes have been made since that time.

5.2.3 Abort Mode Considerations

Work done in the U.S. by Saito (14) explored barrier line sight distance requirements in the context of the abort mode of the passing manoeuvre. He postulates that this may in fact be the critical mode in many cases simply because of the difficulty of aborting and re-integrating the vehicle into a suitable gap in the traffic stream.

His work, based on kinematic modeling efforts, considers that the critical position may in fact exist between the "head and tail" position ,(head of passing car abreast of rear of passed vehicle), and the abreast position , (front bumper abreast of front bumper). By considering a variety of speeds, vehicle positions and both the "car versus car" case and the "car versus truck" case for the abort mode, Saito was able to develop sight distance requirements for aborted passes for different instances.

The results of his work for the case of the passenger car passing a WB-50 truck and aborting from the classic critical position (head to head) are included in Table 2. Values shown have been rounded in the process of metric conversion. The difference between his findings and the current Canadian standards range from 0% at 100 kmh to 9% at 80 khm.

Saito's work seems to reinforce the validity of the current Canadian standards although it must be recognized that it becomes evident that a great deal more work is still needed in this critical area, particularly when substantially longer vehicles are involved (15) (16).

5.3 Conclusions: Passing Operations

The critical case in passing operations is that of two-lane, two-way rural highways. Within this case, from an operational standpoint, the current standards used for Barrier Line Passing Sight Distance have to be looked at critically in respect of their ability to accommodate the proposed change of 2 meters in maximum allowable overall length.

The evidence in the literature indicates that current Canadian standards are significantly higher for Barrier Line PSD than those of both Australia and the United States. In addition, the current Canadian practice compares favorably with Saito's recommendations which do consider both the abort mode and the problem of cars passing trucks. It is evident however that the passing operations problem is still poorly understood and that our current favorable position may in fact be more a result of good engineering judgment rather than any specific technical evidence. While this is not in itself a bad thing, substantial research is required in this area to ensure that the current standards continue to evolve from a solid technical base as both vehicle technology changes and further questions relating to continuing length increases are raised.

6.0 CONCLUSIONS

This report has attempted to evaluate the potential traffic stream effects of a proposed increase in maximum allowable overall length for heavy vehicles from 23 m to 25 m. In reaching a conclusion, a number of factors must be recognized:

* The current 23 m overall length restriction in place in eight of twelve Canadian jurisdictions seems to be working and to be acceptable in the eyes of the public.

* The proposed change is relatively small in nature (2 meters) and would move maximum overall lengths from 23 m to 25 m , constituting about a 9 % change.

* The critical issue upon which the issue turns is that of passing operations on two-lane, two-way highways. Other issues examined in this report were found to be of little or no significance.

* Trucks represent only a minor percentage of the vehicles on 2-lane rural highways. Of the trucks, only a small portion (ie. double combinations) will be able to fully exploit any 25 meter allowance. Other dimensional restrictions proposed for tractor/semi-trailers would make it impossible for a vehicle of this class to exceed 23 meters in length. Tractor-semis currently represent over 85% of the interprovincial truck fleet nationally. This percentage is substantially higher in Atlantic Canada (17). In this context, the impact of an overall length increase to 25 m would seem to be minimal at best.

* The literature shows that both Design and Barrier Line PSD's are relatively insensitive to overall length. Speed is a dominant factor with PSD requirements.

* Current Canadian standards and practice for Barrier Line PSD are considerably more stringent than those used elsewhere and appear to correlate well with recent research results in the U.S. which developed recommendations in consideration of the "car passing truck" case and the possibly more critical abort mode for the passing vehicle.

WHILE A SIGNIFICANT CHANGE IN OVERALL LENGTH ALLOWANCES WOULD REQUIRE A SUBSTANTIVE RESEARCH EFFORT TO SUPPORT, THE CHANGE UNDER CONSIDERATION IS MINOR IN NATURE. GIVEN THIS

FACT AND THE POINTS NOTED ABOVE AND RECOGNIZING THAT ONLY A SMALL PROPORTION OF THE TOTAL LONG-HAUL VEHICLE FLEET WILL EVENTUALLY EXCEED THE CURRENT 23 M ALLOWANCE EVEN IF 25 M IS ALLOWED IT IS RECOMMENDED THAT THE PROPOSED INCREASE GO FORWARD.

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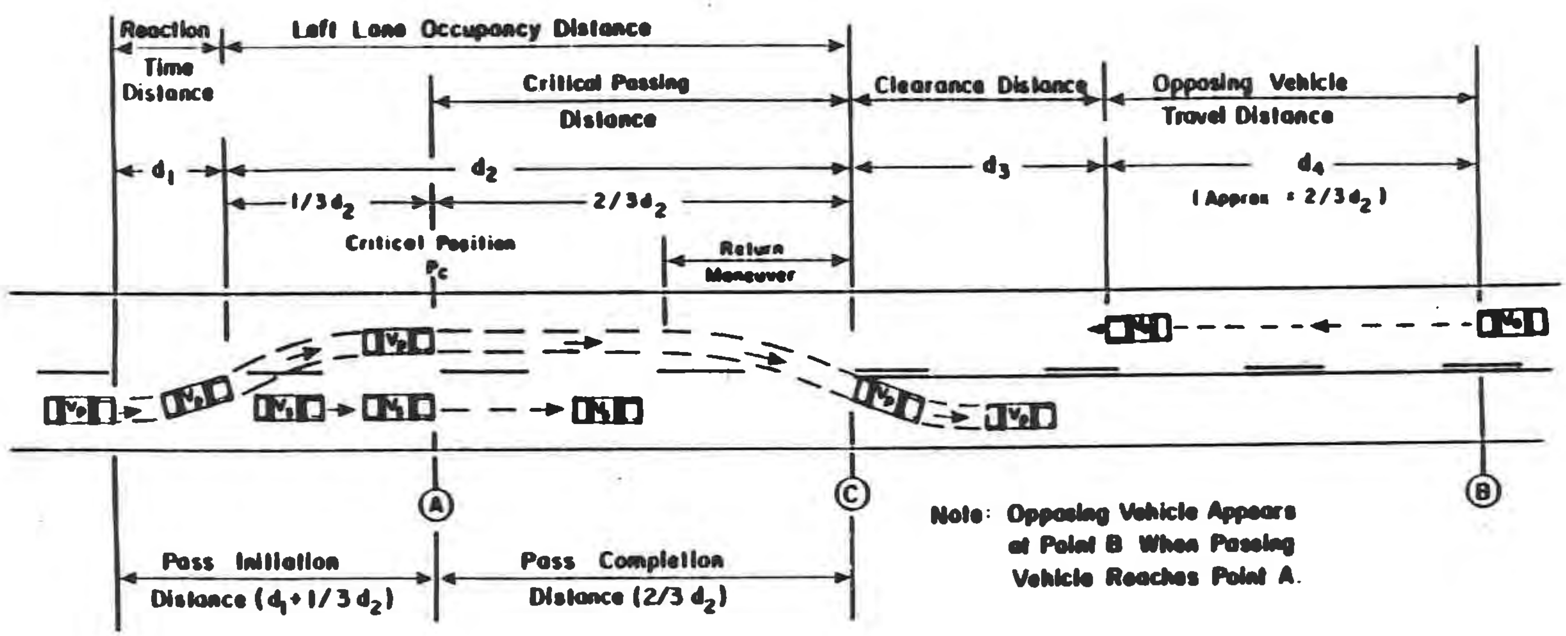
FIGURE 1



Distance Elements Of The Passing Maneuver

TABLE 1

Comparison of Design PSD with Barrier Line PSD

SPEED	DESIGN PSD(1)	Barrier Line PSD(2)
(kmh)	(meters)	(meters)
80	360	275
100	680	400
120	800	565

1: Source: RTAC 1986 Geometric Design Manual

2. Source: RTAC Uniform Traffic Control Devices Manual

TABLE 2

Comparison of Current Barrier Line Practice

(all distances in meters)

Speed (kmh)	Alberta(1)	Canada MUTCDC	U.S. AASHTO	Australian	Quebec Desirable	Quebec Minimum	Saito Research
70		240	200	200	325	250	250
80		275	240	230	375	300	300
90		330	280	260	425	350	350
100	425	400	310	290	500	400	400
110		475	350	320			490

(1) only one value used

(2) values rounded

APPENDIX A

The lane width defined by lane lines normally should be not less than 3.1 m but a minimum of 2.8 m is permissible where roadway width is limited and efficient lane use is important, as at signalized intersections, on bridges and in subways. On sharply curved sections of urban streets lane widths should be increased. At such locations, and also on tangent sections where the roadway width between the directional dividing line and the curb exceeds 6 m the lane line shall be placed to make the lane next to the pavement edge 0.3 to 1 m wider than the adjacent lane.

On multi-lane, relatively high-speed urban arterial streets and on controlled-access highways in urban areas the standards for lane markings shall be the same as those for rural highways.

C2.30 NO-PASSING ZONES

No-passing zones shall be established at vertical or horizontal curves and elsewhere on two and three-lane highways where passing must be prohibited because of dangerously restricted sight distances or other hazardous conditions. No-passing zones shall be marked by a solid barrier line which forms an integral part of the directional dividing line. The barrier line shall be a solid yellow line not less than 10 nor more than 15 cm in width and shall be reflectorized. It shall be separated from the adjoining line by a space equal to one line width.

C2.31 No-Passing Zones on Two-Lane Highways

On a two-lane roadway where the no-passing zone is in one direction only, the normal broken directional dividing line shall be carried through the no-passing zone and a barrier line placed to the right of it.

Where no-passing zones in opposite directions overlap on a two-lane roadway, the directional dividing line shall be replaced by two barrier lines. This marking is illustrated in Figures C.7 and C.10.

C2.32 No-Passing Zones on Three-Lane Highways

No-passing zones shall be marked by a solid yellow barrier line placed to the right of the normal broken lane line. The combination line shall start from the left lane line and shall angle at a slope of not less than 30 to 1 across the centre line to the geometric centre line of the pavement where it shall be continued to the end of the zone. This marking is illustrated in Figures C.10 and C.13.

C2.33 Warrants for No-Passing Zones

The warrant for the establishment of a no-passing zone depends on the 85-percentile speed and the minimum sight distance necessary for safe passing at that speed. In cases where the speed limit is higher than the 85-percentile speed, the speed limit should be used instead of the 85-percentile speed. The higher of these two values is the "speed" to be used in the following table.

A vertical or horizontal curve shall warrant a no-passing zone, and shall be so marked, when the minimum sight distance for the "speed" is equal to or less than that listed as follows:

"Speed" (km/h)	Minimum sight distance for Pavement Markings (m)
50	160
60	200
70	240
80	275
90	330
100	400
110	475
120	565

Sight distance on a vertical curve is the distance at which an object 115 cm above the pavement surface can just be seen from another point 115 cm above the pavement. Similarly, sight distance on a horizontal curve is the distance measured along the centre line (or right-hand lane line of a three-lane highway) between two points 115 cm above the pavement on a line tangent to the embankment or other obstruction that cuts off the view on the inside of the curve. The beginning of a no-passing zone (A_b, B_b in Figures C.6, C.7 and C.8) is that point at which the sight distance first becomes less than that specified in the above table. The end of the zone (A_e, B_e in Figures C.6, C.7 and C.8) is that point at which the sight distance again becomes greater than the minimum specified. In no case shall the resultant barrier line be less than 100 m in length. If the actual no-passing distance is less than 100 m in length, the additional length of barrier line shall be added at the beginning of the zone.

The detailed method of establishing the beginning and end of barrier lines for no-passing zones on vertical curves is shown in Figures C.7 and C.8. The method of establishing and the application of barrier lines for no-passing zones on successive vertical curves is shown in Figure C.9. The application of barrier lines on two-lane and three-lane pavements is shown in Figure C.10 and on two-lane pavements with an added truck lane on up-grades in Figure C.11.

For successive vertical curves (Figure C.9) the beginning and end of each successive barrier line for each direction is established as shown in Figures C.7 and C.8. Where the end of one barrier line in one direction lies within 100 m of the beginning of the next barrier line in the same direction, the barrier line in that direction shall be made continuous over the intervening distance. The detailed method for establishing the beginning and end of barrier lines for no-passing zones on horizontal curves is shown in Figure C.12.

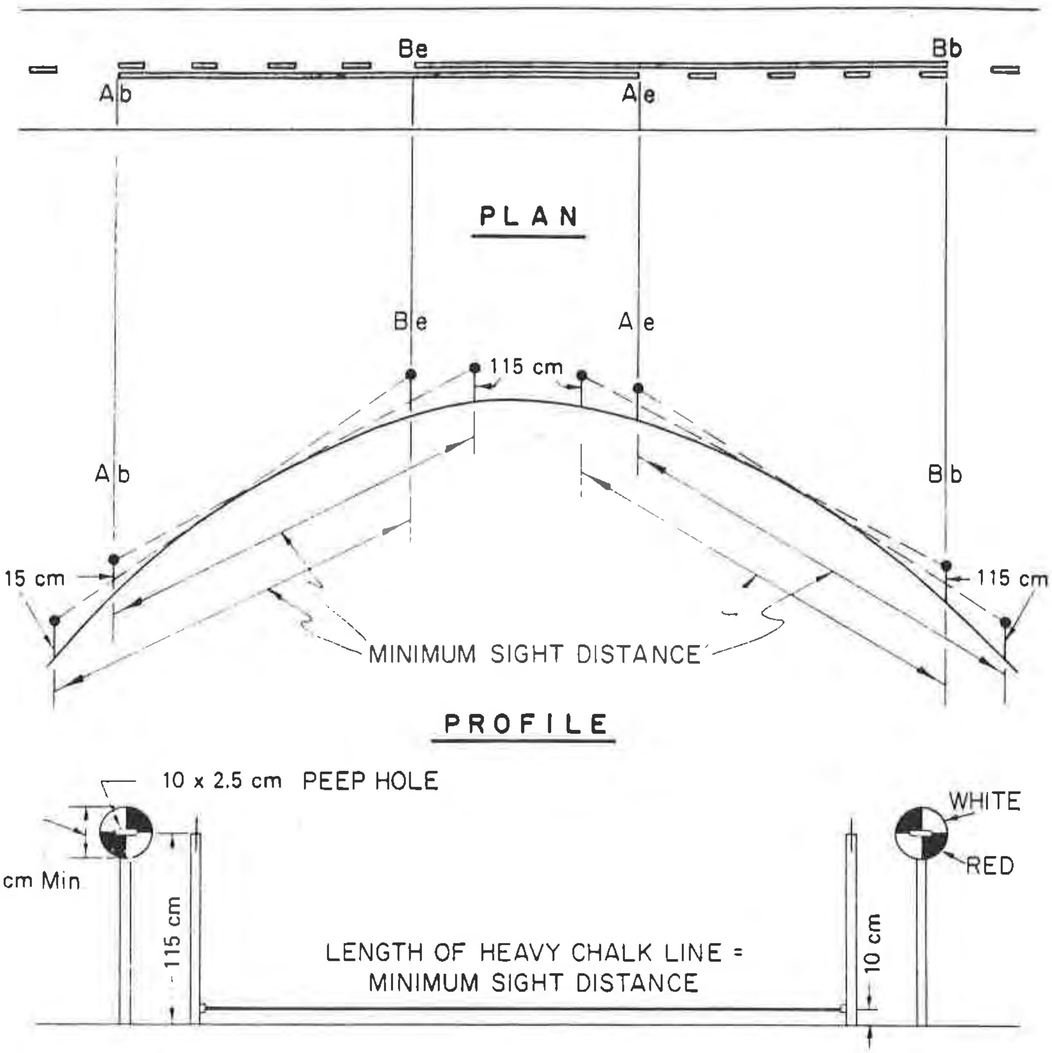
The application of barrier lines for no-passing zones on horizontal curves on two-lane and three-lane pavements is shown in Figure C.13.

On urban streets it is not ordinarily necessary to mark no-passing zones. Speeds are generally low and a normal directional dividing line is usually sufficient. On high-speed arterial streets where no-passing zone markings are required the standards should be the same as for rural highways.

C2.40 LIMITS OF TRAVELLED ROADWAY.

Under certain conditions pavement markings may be used to indicate the limits of the path of travel.

SKETCH SHOWING THE BASIC ELEMENTS USED IN ESTABLISHING NO-PASSING ZONES ON VERTICAL CURVES



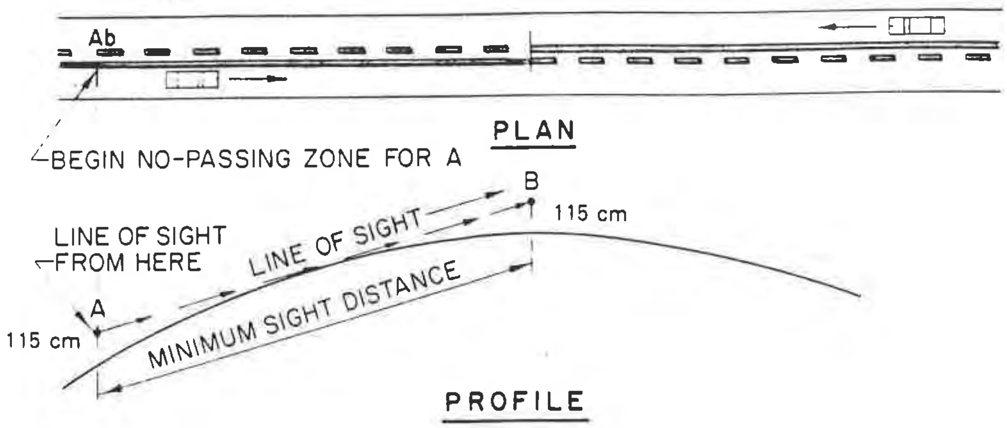
THE ESTABLISHMENT OF THE BEGINNING (Ab, Bb) AND END (Ae, Be) OF BARRIER LINES FOR NO-PASSING ZONES IS ACCOMPLISHED WITH THE AID OF TWO TARGETS AS SHOWN.

THE LENGTH OF THE CHALK LINE JOINING THE TWO TARGETS IN THE LOWER FIGURE IS THE MINIMUM SIGHT DISTANCE FOR PAVEMENT MARKINGS TAKEN FROM THE TABLE IN C2 33

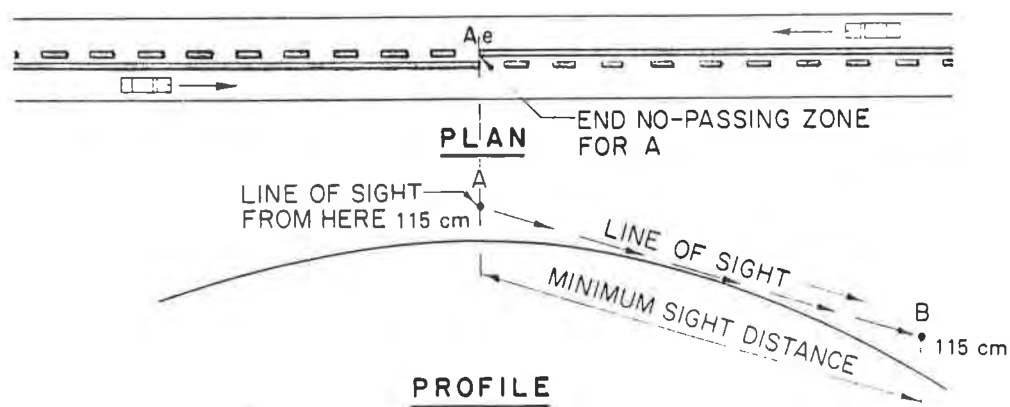
FIGURE C.6

SKETCHES SHOWING THE METHOD OF ESTABLISHING THE TERMINI OF NO-PASSING ZONES ON VERTICAL CURVES

STEP 1 — COMMENCING ON THE UPGRADE SIDE OF THE CURVE WORKMEN A AND B PULL THE LINE TAUT WITH A SIGHTING THROUGH THE SLOT PEEPHOLE IN HIS TARGET TO SEE B'S TARGET. AT THE POINT WHERE THE CENTRE OF B'S TARGET JUST DROPS OUT OF SIGHT, A MARKS A 'T' AT POINT Ab AS SHOWN IN SKETCH BELOW.



STEP 2 — A AND B CONTINUE OVER THE VERTICAL CURVE WITH A SIGHTING B'S TARGET UNTIL THE CENTRE OF B'S TARGET COMES JUST VISIBLE TO A. A THEN MARKS A 'T' AT POINT Ae AS SHOW IN THE SKETCH BELOW.



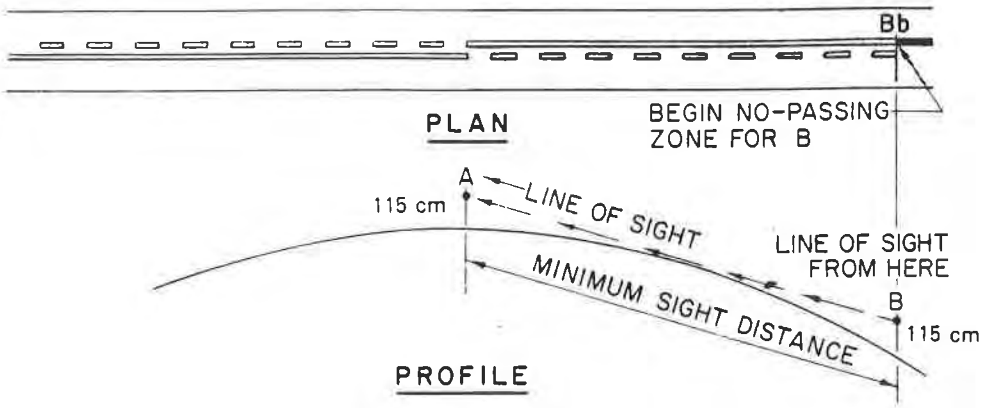
NOTE

THE BEGIN AND END MARKS (⊢ ⊣) SHALL BE PLACED ON THE PAVEMENT SO THAT THEIR STEMS WILL POINT IN THE DIRECTION OF THE SOLID LINE TO BE APPLIED

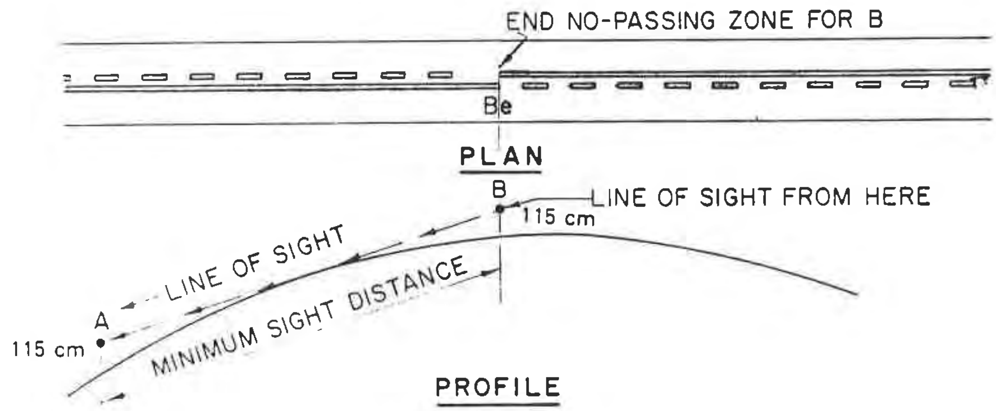
FIGURE C.7

SKETCHES SHOWING THE METHOD OF ESTABLISHING THE TERMINI OF NO-PASSING ZONES ON VERTICAL CURVES

STEP 3 — A AND B AGAIN MOVE AHEAD OVER THE VERTICAL CURVE, THIS TIME IN THE OPPOSITE DIRECTION TO THAT ILLUSTRATED IN FIGURE C.7 COMMENCING AT A POINT ON THE UPGRADE SIDE OF THE CURVE WITH B SIGHTING THROUGH THE SLOT PEEPHOLE IN HIS TARGET TO SEE A'S TARGET. AT THE POINT WHERE THE CENTRE OF A'S TARGET JUST DROPS OUT OF SIGHT, B MARKS A 'T' AT POINT Bb AS SHOWN IN THE SKETCH BELOW.



STEP 4 — A AND B CONTINUE OVER THE VERTICAL CURVE WITH B SIGHTING A'S TARGET UNTIL THE CENTRE OF A'S TARGET BECOMES JUST VISIBLE TO B. B THEN MARKS A 'T' AT POINT Be AS SHOWN IN THE SKETCH BELOW.

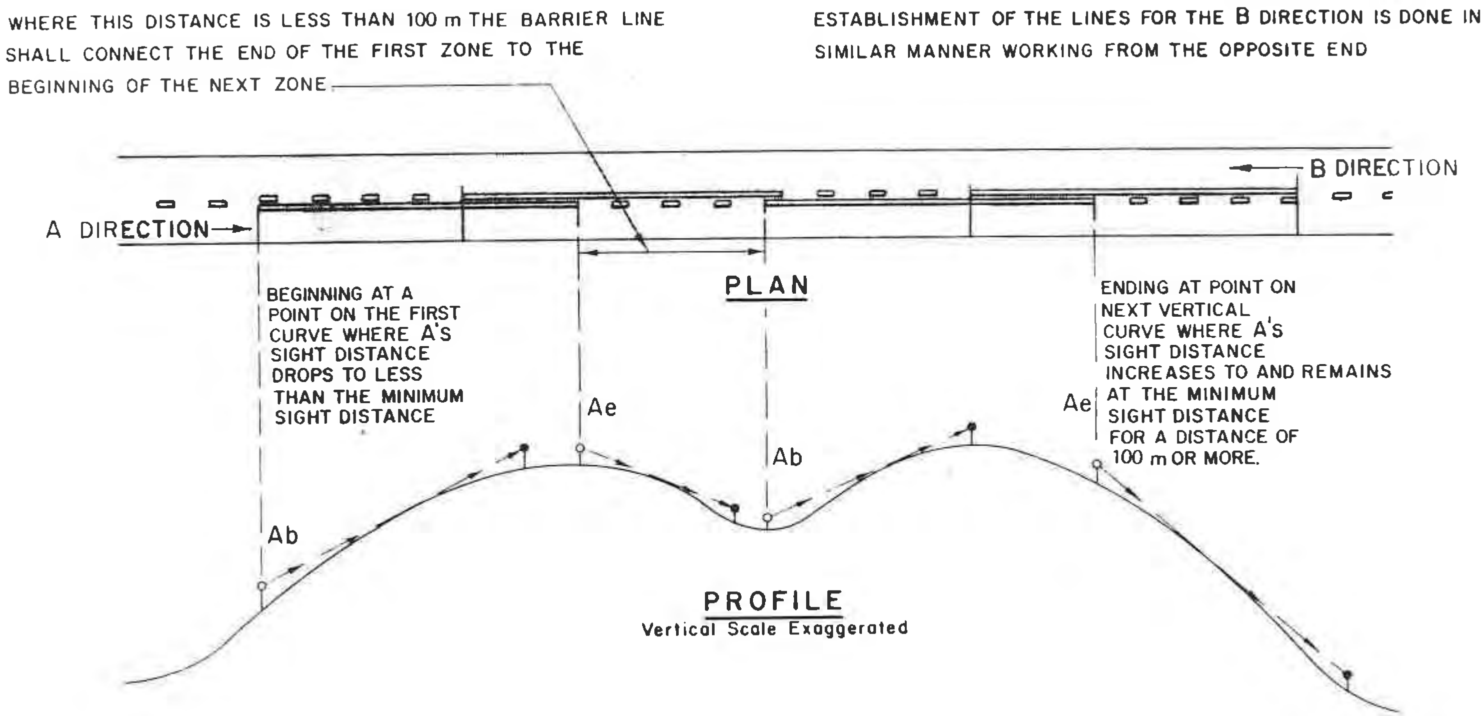


NOTE

IN PRACTICE, IT MAY BE FOUND THAT IT WILL BE MORE EFFICIENT TO VARY THE SEQUENCE OF THE OPERATION IN A MANNER IN WHICH STEPS 1 AND 4 ARE PERFORMED ONE AFTER THE OTHER AND STEPS 2 AND 3 ARE PERFORMED ONE AFTER THE OTHER.

FIGURE C.8

NOTE FOR CLARITY THE ESTABLISHMENT OF THE BEGINNING AND END OF THE BARRIER LINES IN THE A DIRECTION ONLY IS SHOWN. ARROWS INDICATE THE DIRECTION OF THE LINE OF SIGHT. ESTABLISHMENT OF THE LINES FOR THE B DIRECTION IS DONE IN SIMILAR MANNER WORKING FROM THE OPPOSITE END



METHOD OF ESTABLISHING NO-PASSING ZONES FOR SUCCESSIVE VERTICAL CURVES * ON TWO-LANE PAVEMENTS

* SUCCESSIVE HORIZONTAL CURVES ON TWO-LANE PAVEMENTS SHALL BE MARKED SIMILARLY

FIGURE C.9

APPENDIX B

In this section the sight distance required for safe stopping and safe passing is discussed. Criteria for measuring these sight distances and their methods of measurement are discussed in B.3 and B.4.

B.2.2 Stopping sight distance*

It is essential for safe operation that the vehicle operator be able to see far enough ahead to stop if necessary. Conditions that would force a vehicle operator to stop are for example, an object on the roadway, a culvert washout or other fault in the roadway.

Adequate stopping sight distance is required throughout the length of the roadway. Exceptions to this are permitted only in rare cases where the cost of satisfying the requirement would be prohibitive.

Minimum stopping sight distance is the sum of two distances namely:

- *Brake reaction distance*

  The distance travelled during the brake reaction time, that is the time that elapses from the instant an object, for which the driver decides to stop, comes into view to the instant the driver takes remedial action (contacts brake pedal).

- *Braking distance*

  The distance travelled from the time that braking begins to the time the vehicle comes to a stop.

Minimum stopping sight distance values are based on wet pavement surface conditions, in recognition of the lower friction values of wet pavements. In adopting speeds for the purpose of calculating minimum stopping sight distance, wet conditions are assumed to prevail.

For the purpose of calculating minimum sight distance, brake reaction time is taken to be 2.5 s. This value is adopted for all design speeds throughout the range.

Braking distance calculation is based on the laws of motion in which it is assumed that the coefficient of longitudinal friction is constant throughout the braking period.

Values for the coefficient of friction for minimum stopping sight distance assume conditions approaching the worst, and the values adopted are consistent with tires in poor condition operating on a pavement in poor condition with a wet surface. Values for coefficient of friction are taken from A Policy on Geometric Design of Highways and Streets, AASHTO 1984 (Reference 1) and are shown in Appendix A, Table X.B.2.2.

Design values for minimum stopping sight distance for a range of design speeds from 40 km/h to 130 km/h are shown in Table B.2.2a.

When braking occurs on a downgrade, the effect of the grade is to increase the braking distance. Conversely, on an upgrade the effect is to reduce the braking distance. To allow for the effect of grade on minimum stopping sight distance, Table B.2.2b may be applied.

On horizontal curvature, some of the available friction is utilized to produce radial acceleration, reducing the amount available for stopping. This suggests increasing minimum stopping sight distance on horizontal curvature. For most conditions of design speed and radius, adjustment is not required, however, in the middle of the range of design speeds and the smallest radii, some increase is appropriate. The following correction is suggested. For design speeds of 60 km/h to 90 km/h and radii not more than 110% of minimum for the design speed, an increase in minimum stopping sight distance of 5% is appropriate.

Table B.2.2a
Minimum stopping sight distance

design speed km/h	design values for minimum stopping sight distance m
40	45
50	65
60	85
70	110
80	140
90	170
100	200
110	220
120	240
130	260

Provision of adequate stopping sight distance is critical where intersections are located beyond crest vertical curves. Values in Table B.2.2a are often insufficient and decision sight distance given in Table B.2.4 is preferred.

Table B.2.2b
Effect of grade on stopping distance in wet conditions

design speed	correction in stopping distance metres					
	decrease for upgrade			increase for downgrades		
km/h	3%	6%	9%	3%	6%	9%
40	-	-	5	-	-	-
50	5	5	10	-	5	10
60	5	5	10	5	10	15
70	5	10	15	5	10	20
80	10	15	20	10	15	30
90	10	20	25	10	20	40
100	10	20	-	15	30	-
110	15	25	-	15	35	-
120	20	30	-	20	40	-
130	20	30	-	20	40	-

There is evidence that brake reaction time increases for higher speeds by as much as one second, particularly on crest curves. In recognition of this, minimum values are often avoided in favour of longer distances in the design of crest curves.

B.2.3 Passing sight distance*

On two lane roads, vehicles overtaking slower moving vehicles must do so by utilizing lanes normally occupied by opposing traffic. If passing is to be accomplished safely, the driver needs to be able to see a sufficient distance ahead to be satisfied that there is a length of road clear of opposing traffic to complete the passing manoeuvre without cutting off the passed vehicle in advance of meeting an opposing vehicle appearing during the manouevre. When required, a driver can return to the right-hand lane without passing if he sees opposing traffic is too close when the manouevre is incomplete. Passing manouevres made without the driver able to see a safe passing distance ahead, do not have an acceptable level of safety.

Passing sight distance for use in design is determined on the basis of the length needed to safely complete normal passing manoeuvres. While there may be occasions to consider multiple passings, where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum standards. Instead, sight distance is determined for a single vehicle passing a single vehicle. Longer sight distances occur in design and these locations can accommodate an occasional multiple passing.

Standard minimum passing sight distance values are given in Table B.2.3 for a range of design speed from 50 km/h to 130 km/h. It is important, for reasons of safety and service, to provide as many passing opportunities as possible in each section of road. It is preferable to avoid a long stretch where passing is not possible. The amount of passing sight distance available on a section of road has considerable influence on the average running speed of the traffic. This is particularly true where a road is operating near capacity. The economic effects of reduced speeds cannot be accurately determined, but there is no doubt that road users benefit considerably when able to operate at or near the design speed with minimum interference from other vehicles. These economic effects should be considered when setting vertical alignment.

Table B.2.3
Minimum passing sight distance

design speed km/h	minimum passing sight distance m
50	340
60	420
70	480
80	560
90	620
100	680
110	740
120	800
130	860

B.2.4 Decision sight distance*

Minimum stopping sight distance is usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under ordinary circumstances. However, this distance is often inadequate when drivers must make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual manoeuvres are required . Limiting sight distance to that provided for stopping may also preclude drivers from performing evasive manoeuvres, which are often less hazardous and otherwise preferable to stopping. Even with an appropriate complement of standard traffic control devices, stopping sight distance might not provide sufficient visibility distance for drivers to corroborate advance warnings and to perform the necessary manoeuvres. It is evident that there are many locations where it would be prudent to provide longer sight distance. In these circumstances, the use of decision sight distance instead of minimum stopping sight distance provides the greater length that drivers need.

Decision sight distance is the distance required for a driver to detect an information source or hazard which is difficult to perceive in a roadway environment that might be visually cluttered, recognize the hazard or its threat potential, select appropriate action and complete the manoeuvre safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to manoeuvre their vehicles at the same or reduced speed rather than simply to stop, it is substantially greater than minimum stopping sight distance.

Drivers need decision sight distance whenever there is a likelihood for error in either information reception, decision making, or control actions. Examples of critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance are: interchanges and intersections; locations where unusual or unexpected manoeuvres are required; changes in cross section such as toll plazas and lane drops; and areas of concentrated demand where sources of information compete, for example from roadway elements, traffic, traffic control devices, and advertising signs.

The decision sight distances in Table B.2.4 are used for appropriate sight distance at critical locations and serve as criteria in evaluating the suitability of the sight lengths at these locations. Because of the additional safety and manoeuvreability these lengths yield, decision sight distances instead of minimum stopping sight distances are provided at critical locations. If it is not feasible to provide these distances because of horizontal or vertical curvature, special attention should be given to the use of suitable traffic control devices for providing advance warning of the conditions that are likely to be encountered.

A range of decision sight distance values applicable to most situations has been developed. The range recognizes the variation in complexity that occurs at various sites. For less complex situations, values toward the lower end of the range are appropriate and for more complexity, values at the upper end are used.

Table B.2.4
Decision sight distance

design speed km/h	decision sight distance m
40	110-160
50	140-190
60	170-230
70	200-270
80	230-310
90	280-360
100	300-390
110	330-430
120	360-470
130	390-500

APPENDIX C

The reduced longitudinal friction has the effect of increasing the braking distance by 9.0 m, giving a total calculated minimum stopping sight distance of 145.6 m. This exceeds the reduced values for design by 5.6 m or 4%.

The results of similar calculation for minimum radius curves and a range of design speeds are shown in Table X.B.2.2b.

By comparing Table X.B.2.2a and Table X.B.2.2b it is readily apparent that the range of design speeds from 60 km/h to 90 km/h is the critical region for stopping sight distance on curves.

Because the required sight distance on a horizontal curve is an important parameter in the calculation of the minimum clearance from the edge of pavement to an obstruction, roadside clearances were checked using the stopping sight distances in Table X.B.2.2b to determine if they differed significantly from current design values (Figure B.3.3.3a).

The formula for calculating the required side clearance is

$$C = R\left(1 - \cos\left(\frac{28.65S}{R}\right)\right)$$

In this comparison, for a given radius curve, the parameter S could be either the design stopping sight distance from Table X.B.2.2a or the calculated stopping sight distance from Table X.B.2.2b. The results are shown in Table X.B.2.2c

The results of this calculation indicate that the range of design speeds 60 km/h to 90 km/h is critical. The design clearance is deficient by up to 11%.

For these reasons this following adjustment is suggested:

For design speeds of 60 km/h to 90 km/h, and radius not exceeding 110% of minimum stopping sight distance is increased by 5%.

X.B.2.3 Passing sight distance

Passing sight distance for use in design is determined on the basis of the length needed to safely complete normal passing manoeuvres. While there may be occasions to consider multiple passings, where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum standards. Instead, sight distance is determined for a single vehicle passing a single vehicle. Longer sight distances occur in design and these locations can accommodate an occassional mulitiple passing.

In determining minimum passing sight distances on two-lane highways for design use, certain assumptions for traffic behaviour are necessary, some of which offer a wide choice. The assumed control for driver behaviour should be that practiced by a high percentage of drivers, rather than the average driver. The following assumptions are made:

- The overtaken vehicle travels at uniform speed.
- The passing vehicle has reduced speed and trails the overtaken vehicle as it enters a passing section.
- When the passing section is reached, the driver requires a short period of time to perceive the clear passing section and to react to start his manoeuvre.
- Passing is accomplished under what may be termed a delayed start and a hurried return in the face of opposing traffic. The passing vehicle accelerates during the manoeuvre, and its average speed during the occupancy of the left lane is 15 km/h higher than that of the overtaken vehicle.
- When the passing vehicle returns to its lane, there is a suitable clearance length between it and an oncoming vehicle in the other lane.

Some drivers accelerate at the beginning of a passing manoeuvre to an appreciably higher speed and then continue at a uniform speed until the passing is completed. Many drivers accelerate at a fairly high rate until just beyond the vehicle being passed and then complete the manoeuvre either without further acceleration or at reduced speed. For simplicity, extraordinary manoeuvres are ignored and passing distances are developed with the use of observed speeds and times that fit the practices of a high percentage of drivers.

The minimum passing sight distance for two-lane highways is determined as the sum of the four distances shown in Figure X.B.2.3.

Table X.B.2.2b
Calculated stopping sight distance on minimum radius curves

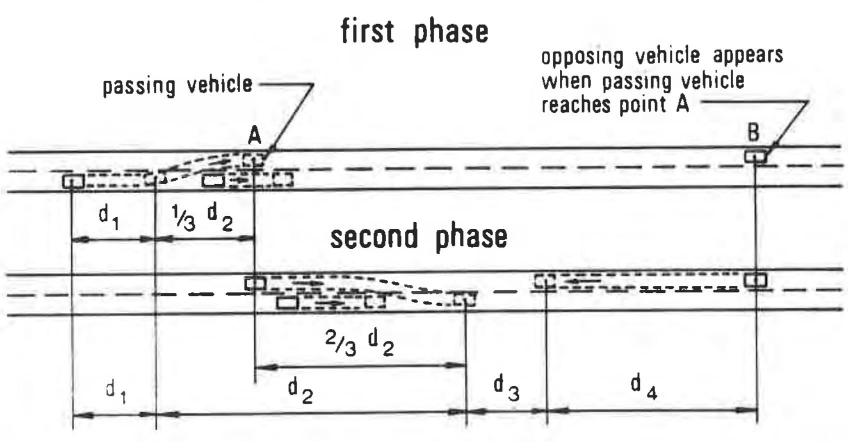
design speed km/h	calculated stopping sight distance m $e_{max} = 0.04$	$e_{max} = 0.06$	$e_{max} = 0.08$
40	46.2 +	–	–
50	64.9	65.0	65.4
60	87.9 +	88.6 +	88.4 +
70	116.9 +	115.7 +	116.1 +
80	146.4 +	146.6 +	146.2 +
90	179.6 +	179.5 +	180.5 +
100	202.7 +	202.1 +	202.6 +
110	–	220.1	219.8
120	–	239.7	239.0
130	–	257.6	257.1

+ indicates the calculated value is greater than the corresponding design value in Table X.B.2.2a

Table X.B.2.2c
Required side clearance on minimum radius curves

design speed km/h	e_{max}	required side clearance calculated m	required side clearance design m	% difference
40	0.04	4.39	4.17	5.1
50	0.04	5.06	5.24	-3.4
60	0.04	6.39	5.98	6.4
70	0.04	8.48	7.52	11.4
80	0.04	9.51	8.71	8.5
90	0.04	10.56	9.47	10.4
100	0.04	10.45	10.17	2.6
50	0.06	5.80	5.81	0.0
60	0.06	7.48	6.89	7.9
70	0.06	8.74	7.91	9.5
80	0.06	10.67	9.74	8.7
90	0.06	11.78	10.57	10.2
100	0.06	11.55	11.32	2.1
110	0.06	10.07	10.06	0.0
120	0.06	9.56	9.58	0.0
130	0.06	8.72	8.88	-1.9
50	0.08	6.59	6.51	1.2
60	0.08	8.05	7.45	7.4
70	0.08	9.81	8.82	10.1
80	0.08	11.52	10.57	8.2
90	0.08	13.47	11.96	11.2
100	0.08	13.08	12.75	2.5
110	0.08	11.35	11.38	0.0
120	0.08	10.63	10.72	-1.0
130	0.08	9.94	10.16	-2.0

Figure X.B.2.3
Elements of passing sight distance



d_1 Distance travelled during perception and reaction time and during the initial acceleration to the point of encroachment of the left lane.

d_2 Distance travelled while the passing vehicle occupies the left lane.

d_3 Distance between the passing vehicle at the end of its manoeuvre and the opposing vehicle.

d_4 Distance travelled by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or $^2/_3$ of d_2 above.

X.B.2.4 Decision sight distance

Decision sight distance is the distance required for a driver to detect an information source or hazard which is difficult to perceive in a roadway environment that might be visually cluttered, recognize the hazard or its threat potential, select appropriate action and complete the manoeuvre safely and efficiently. Because decision sight distance gives drivers additional margin for error and affords them sufficient length to manoeuvre their vehicles at the same or reduced speed rather than simply to stop, it is substantially greater than stopping sight distance.

Decision sight distance values that will be applicable to most situations has been developed. The range has been provided in recognition of the variation in complexity that may exist at various sites.

The values were analytically derived from a summation of pre-manoeuvre and manoeuvre times converted into distance and empirically validated at a number of locations. Table X.B.2.4 shows these values, rounded for design, along with the factors used to compute the distances.

Pre-manoeuvre time is the time required for a driver to process information relative to a hazard. It consists of the times to detect and recognize the hazard, to decide on proper manoeuvres and initiate required action. This process thus includes both reaction and response initiation.

Detection and recognition times are two elements of the information-handling process and include time periods for latency (the delay between the time a hazard is presented and the time that the eyes begin to move), eye movement to hazard, eye fixation, and finally, recognition and perception of the hazard. Times for these elements increase with the complexity and number of signals (hazards) and with increasing speed. Times up to 3 s have been reported.

Once the hazard is perceived, the next steps in the process are to identify the alternative manoeuvres, select one, and then initiate the required action. Because the required manoeuvre is likely to be a lane change, the decision and response initiation times, the time to decide on this manoeuvre, search for gaps, and initiate the action can range from 2.0 s to 7.1 s or even longer under heavy traffic volumes.

The final step is calculation of the manoeuvre time, the time required to accomplish a vehicle manoeuvre. Because the intent of decision sight distance is to allow drivers time to take an evasive action other than a hurried stop, the assumption is made that a lane change is representative of manoeuvres performed. On the basis of data derived for passing sight distance, this time is assumed to be between 3.5 s and 4.5 s decreasing with increasing speed.

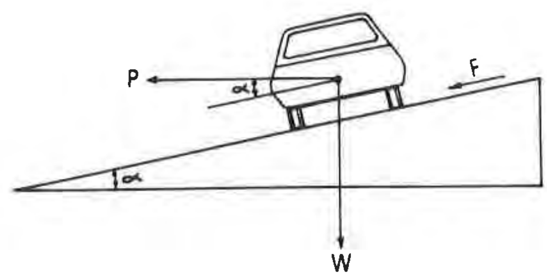
The calculations for decision sight distance based on the above assumptions are given in Table X.B.2.4.

X.B.3.1.1 Speed-radius relationship

When a vehicle is travelling along a circular curve at a constant speed, it is experiencing an acceleration towards the centre of the circle. The centripetal force providing this acceleration is the friction between tire and pavement and if the travelled road is superelevated, the friction force is supplemented by a component of the force of gravity due to the weight of the car.

Figure X.B.3.1.1 illustrates the dynamics of a vehicle travelling on a circular curve of instant radius at constant speed. The centripetal force, acting on the vehicle toward the centre of the circle, produces radial acceleration.

Figure X.B.3.1.1
Dynamics of vehicle on circular curve



W Weight of vehicle
M Mass of vehicle
P Centripetal force (horizontal)
F Friction force between tires and road surface (parallel to road surface)
α angle of superelevation ($\tan \alpha = e$)
V speed of vehicle
R Radius of curve

Table X.B.2.4
Calculation of decision sight distance for design

design speed km/h	time: pre-manoeuvre time: detection and recognition s	time: pre-manoeuvre time: decision response and initiation s	time: manoeuvre (lane change) s	time: total s	decision sight distance: calculated m	decision sight distance: rounded for design m
40	1.5-3.0	4.2-6.5	4.5	10.2-14.0	113-155	110-160
50	1.5-3.0	4.2-6.5	4.5	10.2-14.0	141-194	140-190
60	1.5-3.0	4.2-6.5	4.5	10.2-14.0	170-233	170-230
70	1.5-3.0	4.2-6.5	4.5	10.2-14.0	198-272	200-270
80	1.5-3.0	4.2-6.5	4.5	10.2-14.0	226-311	230-310
90	2.0-3.0	4.7-7.0	4.5	11.2-14.5	280-362	280-360
100	2.0-3.0	4.7-7.0	4.0	10.7-14.0	297-389	300-390
110	2.0-3.0	4.7-7.0	4.0	10.7-14.0	327-427	330-430
120	2.0-3.0	4.7-7.0	4.0	10.7-14.0	357-466	360-470
130	2.0-3.0	4.7-7.0	4.0	10.7-14.0	386-505	390-500

APPENDIX D

The opposing vehicle is assumed to be traveling at the same speed as the passing vehicle, so $d_4 = 2d_2/3$. The distance d_4 is given in Table III-4 and shown on Figure III-2.

Design Values

The "total" curve in Figure III-2 is determined by the sum of the named elements. For each passing speed, it indicates the minimum passing sight distance for a vehicle passing another vehicle traveling 10 mph slower, in the face of an opposing vehicle traveling at the same speed as the passing vehicle. On determination of a likely and logical relation between average passing speed and the highway design speed, these distances can be used to express the minimum passing sight distance needed for design purposes.

The ranges of speeds of the passed and passing vehicles are affected by traffic volume. When traffic volume is low (level-of-service A), there are few vehicles that need to be passed, but as volume increases (level of service decreases) and the need to pass increases, the larger volume of oncoming traffic decreases the passing opportunities. At high volume (level-of-service D or lower) there are few, if any, passing opportunities. The speed of the passed vehicle has been assumed to be the average running speed at a traffic volume near capacity as represented by the curve for intermediate volumes in Figure II-19. The speed of the passing vehicle is assumed to be 10 mph greater. The assumed speeds for passing vehicles in Table III-5 represent the likely passing speeds on two-lane highways. Passing sight distances for these passing speeds would accommodate the majority of the desired passing maneuvers. They correspond to the "total" curve of Figure III-2. For convenience, a design speed grid has been superimposed on the upper half of the figure, permitting direct reading of the passing sight distance in terms of the design speed. The rounded values in the last column of Table III-5 are design values for minimum passing sight distance. In designing a highway these distances should be exceeded as much as practicable and such sections provided as often as can be done with reasonable costs so as to present as many passing opportunities as possible.

These minimum passing sight distances for design should not be confused with other distances used as the warrants for placing no-passing zone pavement stripes on completed highways. Such values as shown in section 3B-5 of the MUTCD (8) are substantially less than

design distances and are derived for traffic operating-control needs that are based on different assumptions from those for highway design.

Design Speed (mph)	Assumed Speeds: Passed Vehicle (mph)	Assumed Speeds: Passing Vehicle (mph)	Minimum Passing Sight Distance (ft): Figure III-2	Minimum Passing Sight Distance (ft): Rounded
20	20	30	810	800
30	26	36	1,090	1,100
40	34	44	1,480	1,500
50	41	51	1,840	1,800
60	47	57	2,140	2,100
65	50	60	2,310	2,300
70	54	64	2,490	2,500

Table III-5. Minimum passing sight distance for design of two-lane highways.

Effect of Grade on Passing Sight Distance

Appreciable grades increase the sight distance required for safe passing. Passing is easier for the vehicle traveling downgrade because the overtaking vehicle can accelerate more rapidly than on the level and thus can reduce the time of passing, but the overtaken vehicle can also accelerate easily so that a dangerous situation akin to a racing contest may result.

The sight distances required to permit vehicles traveling upgrade to pass with safety are greater than those required on level roads because of reduced acceleration of the passing vehicle (which increases the time of passing) and the likelihood of opposing traffic speeding up (which increases the distance traveled by it). Compensating for this somewhat are the factors that the passed vehicle frequently is a truck that usually loses some speed on appreciable upgrades and that many drivers are aware of the greater distances needed for passing upgrade compared with level conditions.

If passings are to be performed safely on upgrades, the passing sight distance should be greater than the derived minimum. Specific adjustments for design use are unavailable, but the designer should recognize the desirability of increasing the minimum shown in Table III-5.

Frequency and Length of Passing Sections

Sight distance adequate for passing should be encountered frequently on two-lane highways, and at each passing section the length of roadway with sight distance ahead equal to or greater than the minimum passing sight distance should be as long as feasible. Frequency and length of passing sections for highways depend principally on the topography, the design speed of highway, and the cost; for streets, the spacing of intersections is the principal consideration. On roadways with high volumes that approach capacity frequent and long passing sections are essential. On roadways with intermediate to low volumes the need is not as great, but passing sections are still an important adjunct for efficiency and safety.

It is not possible to directly indicate the frequency with which passing sections should be provided on two-lane highways due to the physical and cost limitations. On almost all roads and selected streets some passing sections are provided in the normal course of design, but the designer's appreciation of their importance and his studied attempt to provide them usually can ensure others at little or no additional cost. On some two-lane roads in steep mountainous terrain, it may be more economical to build intermittent four-lane sections with stopping sight distance in lieu of two-lane sections with passing sight distance.

The minimum passing sight distance is sufficient for a single or isolated passing only. Design with only minimum sight distance will not assure that safe passings can be made. Even on low-volume roadways a driver desiring to pass may, on reaching the passing section, find vehicles in the opposing lane and thus be unable to use the section or at least not be able to begin to pass at once.

The importance of frequent passing sections is illustrated by their effect on the service volume of a two-lane, two-way highway. Table 10.7 of the HCM (13) shows, for example, that, for an average operating speed of 50 mph over a high-type two-lane highway (70-mph AHS, 12-ft lanes), the service volume is reduced from 900 passenger cars per hour where there are no sight restrictions to about 680 passenger cars per hour where passing sight distance is available on

only 40 percent of the highway. The effect of restricted sight distance on service volume is even more severe where a design speed lower than 70 mph is assumed as a basis for design.

These data indicate another possible criterion for passing sight distance design on two-lane highways. On sections of roadway that are several miles or more in length, the available sight distances along this length can be summarized to show the percentage of length with sight distance greater than the passing minimum. Analysis of capacity related to this percentage would indicate whether or not alinement and profile adjustments are necessary to accommodate the DHV's. When highway sight distances are analyzed over the whole range of lengths within which passings are made, a new design criterion may be evaluated. Where high traffic volumes are expected on a highway and a high level of capacity is to be maintained, it is virtually mandatory that frequent or nearly continuous passing sight distances are provided.

Sight Distance for Multilane Highways

It is not necessary to consider passing sight distance on highways or streets that have two or more traffic lanes in each direction of travel. Passing maneuvers on multilane roadways are expected to occur within the limits of each one-way traveled way. Thus passing maneuvers that require crossing the centerline of four-lane undivided roadways or crossing the median of four-lane divided roadways are reckless and should be prohibited.

It is imperative that multilane roadways have continuously adequate stopping sight distance, with greater than minimum distance being preferred. Lengths of stopping sight distance vary with vehicle speed and are discussed in detail at the beginning of this chapter.

Criteria for Measuring Sight Distance

Sight distance is the distance along a roadway that an object of specified height is continuously visible to the driver. This distance is dependent on the height of the driver's eye above the road surface, the specified object height above the road surface, and the height of sight obstructions within the line of sight.

APPENDIX E

TABLE 19–5
Minimum Stopping Sight Distance on Wet Pavements*

Design Speed mph	Design Speed km/h	Assumed Speed for Condition mph (km/h)	Brake Reaction Time (s)	Brake Reaction Distance (ft)	Coefficient of Friction, f	Braking Distance on Level (ft)	Stopping Sight Distance Computed (ft)	Stopping Sight Distance Rounded for Design (ft)
20	30	20 (30)	2.5	73	0.40	33	106	120
30	50	28 (45)–30 (50)	2.5	103–110	0.35	75–86	178–196	200–200
40	65	36 (55)–40 (65)	2.5	132–147	0.32	135–167	267–314	275–325
50	80	44 (70)–50 (80)	2.5	161–183	0.30	215–278	376–461	375–475
60	95	52 (85)–60 (95)	2.5	191–220	0 29	311–414	502–634	525–650
65	105	55 (90)–65 (105)	2.5	202–238	0.29	348–486	550–724	550–725
70	115	58 (95)–70 (115)	2.5	213–257	0.28	400–583	613–840	625–850
75	120	61 (100)–75 (120)	2.5	224–275	0 28	443–670	667–945	675–950
80	130	64 (105)–80 (130)	2.5	235–293	0.27	506–790	741–1083	750–1100

Metric conversion factor: multiply value by 0.305 m/ft.

TABLE 19–6
Decision Sight Distance*

Design Speed mph	Design Speed km/h	Times (s) Premaneuver: Detection and Recognition	Premaneuver: Decision and Response Initiation	Maneuver (lane change)	Summation	Decision Sight Distance (ft) Computed	Rounded for Design
30	50	1.5–3	4.2–6.5	4.5	10.2–14	449–616	450–625
40	65	1.5–3	4.2–6.5	4.5	10.2–14	598–821	600–825
50	80	1.5–3	4.2–6.5	4.5	10.2–14	748–1027	750–1025
60	95	2–3	4.7–7.0	4.5	11.2–14.5	986–1276	1000–1275
70	115	2–3	4.7–7.0	4.0	10.7–14	1098–1437	1100–1450
80	130	2–3	4.7–7.0	4.0	10.7–14	1255–1643	1250–1650

*Metric conversion factor: multiply value by 0.305 m/ft.

SOURCE: McGee, H. W., Moore, W., Knapp, B. G., and Sanders, J. H. *Decision Sight Distance for Highway Design and Traffic Requirements*, U.S. Department of Transportation, FHWA, Washington, D.C. 1978.

distance at which drivers can detect a signal or hazard in a cluttered or visually noisy roadway environment, recognize it, and perform the required actions safely. Its values are substantially longer than those for stopping sight distance.

Locations where it is desirable to provide decision sight distance are: (1) complex interchanges and intersections; (2) any locations where unusual or unexpected maneuvers are required; (3) any variation in cross sections, such as toll plazas and lane drops; (4) where roadway elements, traffic and signs, signals, and other traffic control devices compete; and (5) areas where an unexpected maneuver may be required.

Table 19-6 shows a range of decision sight distances based on most complex situations. In measuring decision sight distance, the 3.5-ft (1.05-m) seated eye height criterion used to measure stopping sight distance is retained. However, the 6-in. (15 cm) object height is not retained and a zero height of object is adopted. Table 19-6 also shows the factors used to compute decision sight distances.

Passing sight distance. Passing sight distance is applicable only on two-lane, two-way highways. Passing sight distance is the length of highway ahead necessary for one vehicle to pass another before meeting an opposing vehicle which might appear after the pass began. Passing sight distances used for design, given in Table 19-7, are based on various traffic behavior assumptions.[13]

TABLE 19–7
Minimum Passing Sight Distances

Used for Design: Design Speed mph	km/h	Minimum Passing Sight Distance ft	m	Used for Pavement Marking: 85th Percentile Speed mph	km/h	Minimum Passing Sight Distance ft	m
20	30	800	245	—	—	—	—
30	50	1100	335	30	48	500	152
40	64	1500	457	40	64	600	183
50	80	1800	549	50	80	800	244
60	97	2100	640	60	97	1000	305
65	105	2300	701	—	—	—	—
70	113	2500	762	70	113	1200	366
75	121	2600	793	—	—	—	—
80	129	2700	823	—	—	—	—

Passing sight distances for purposes of pavement marking are also given in Table 19-7. No-passing zone markings, given in the *Manual on Uniform Traffic Control Devices*,[14]

[13] "A Policy on Design of Rural Highways." pp. 140–145. Also refer to the new AASHTO policy on rural and urban highways when it is published.

[14] FEDERAL HIGHWAY ADMINISTRATION, U.S. DEPARTMENT OF TRANSPORTATION, *Manual on Uniform Traffic Control Devices for Streets and Highways*, (Washington, D.C.: Government Printing Office, 1978), p. 3B-8.

are based on different assumptions which result in lower values. No-passing zones are based on the 85th percentile speed during low-volume conditions, which is slightly less than the design speed.

Sight distance adequate for passing should be provided frequently in design of two-lane highways, and each passing section should be as long as feasible. Although the frequency and lengths of such passing sections depend on physical and cost considerations and cannot be reduced to a standard, the importance of providing passing opportunities on as much of the length of a two-lane highway as possible cannot be overemphasized. The percentage of the highway where passing can take place affects not only capacity, but also the safety, comfort, and convenience of all highway users.

For purposes of design, passing sight distance for both horizontal and vertical restrictions is measured from a "seeing" height of 3.5 ft (1.05 m) to an object height of 4.25 ft (1.3 m). For purposes of marking pavement, it is measured from a "seeing" height of 3.75 ft (1.15 m) to an object height of 3.75 ft (1.15 m).

Intersection sight distance. Intersections should be planned and located to provide as much sight distance as possible. In achieving a safe highway design, as a minimum, there should be sufficient sight distance for the driver on the minor highway to cross the major highway without requiring approaching traffic to reduce speed. Minimums for different design speeds are shown in Table 19-8. Stop controls are assumed; other forms of traffic control have different intersection sight distance requirements.

TABLE 19-8
Suggested Corner Sight Distance at Intersections*

Design speed mph (km/h)				
20 (32)	30 (48)	40 (64)	50 (80)	60 (97)
Minimum corner intersection sight distance* ft (m)				
200 (61)	300 (91)	400 (122)	500 (152)	600 (183)

*Corner sight distance measured from a point of the minor road at least 15 ft (4.6 m) from the edge of the major road pavement and measured from a height of eye of 3.5 ft (1.05 m) on the minor road to a height of object of 4.25 ft (1.3 m) on the major road.

Procedures for checking plans. It is often desirable during the preliminary design stage to determine graphically the sight distances and record them at frequent intervals. Methods for scaling sight distances and a typical sight distance record which should be shown on final plans are shown in Figure 19.2. For two-lane highways, passing sight distance, in addition to stopping sight distance, should be shown.

Horizontal sight distance on the inside of curves may be limited by obstructions such as buildings, plant growth, or cut slope. Horizontal sight distance is measured along a straight edge, as indicated in the upper left in Figure 19.2.

Figure 19.2. Scaling and recording sight distances on plans. (Metric conversion factor: multiply values by 0.305 m/ft.) SOURCE: Adapted from *A Policy on Geometric Design of Rural Highways*, Washington, D.C.: American Association of State Highway Officials, 1965, p. 150.