7.0 SHOULDER AND LANE WIDTHS

7.1 GENERAL

The literature generally defines a shoulder as that portion of the roadway which has been constructed to support the emergency movement and stopping of vehicles, but not as part of the normal travelled-way for vehicles. The shoulder does not include any berm, verge, rounding, or extra width provided to accommodate guide posts, guard fence etc. Historically the shoulder, although constructed as an integral part of the road base, has not until recent years been sealed to provide the same quality of surface as the traffic lanes.

The major functions of shoulders according to Armour and McLean (1983) are:

(a) Shoulders are a structural element of the total pavement, providing lateral support to the traffic lanes.

(b) Shoulders allow construction related edge effects to be located away from the trafficked section of the pavement.

(c) Shoulders serve to drain water away from the trafficked section of the pavement.

(d) Shoulders tend to increase the 'effective' width of the traffic lanes, and so increase lateral clearances.

(e) Shoulders provide a recovery area for errant vehicles.

(f) Shoulders provide space for slower vehicles to allow faster vehicles to pass.

(g) Shoulders allow moving vehicles to pass vehicles disabled in the traffic lane.

(h) Shoulders allow moving vehicles to pass vehicles turning right from the traffic lane.

(i) Wide shoulders enable a stopped vehicle to stand clear of the traffic lanes.

Although the effectiveness of shoulders on two lane rural roads are considered separately here from other road design features their effectiveness can not be entirely separated from other features. For instance, the horizontal and vertical alignment of a particular road may dictate that wider shoulders increase sight distance.

The majority of the earlier research has been conducted in the United States with some conflicting results being evident. However, in recent years more Australian research has been conducted, mainly by the Australian Road Research Board.

7.2 LANE WIDTH

The effect on the crash rates of varying lane widths, especially at traffic volumes of less than 5,000 vehicles per day appears to be small and Roy Jorgensen Associates (1978) concluded that although crash rates rise for lane widths below eleven feet (3.35 metres) there is no rise or fall when lane widths were increased above eleven feet.

Where shoulders are paved it is probably more sensible not to try and separate the effect of lane width and shoulder width, but to group them together as suggested by McLean (1985).

7.3 TYPE OF SHOULDER SURFACE

Roy Jorgensen Associates (1978) produced base crash rates for roads with paved and unpaved shoulders with and without curves of 600 metres radius and in various average daily traffic groups as shown in Table 7.1 as quoted by McLean (1985).

Table 7.1

Base Crash Rates
Injury Crashes per Million Vehicle Kilometres

ADT Group	Radius of Curvature	Shoulder Types Unpaved	Shoulder Types Paved
0-999	GT 600 m	1.15	0.90
	LE 600 m	1.33	1.04
1000-2499	GT 600 m	0.77	0.60
	LE 600 m	0.89	0.69
2500-4999	GT 600 m	0.78	0.61
	LE 600 m	0.89	0.70
GT 5000	GT 600 m	0.82	0.64
	LE 600 m	0.94	0.74

Source: Roy Jorgensen Associates (1978)

Sealed shoulders are usually provided on roads with higher volumes so that a comparison between roads with and without shoulders may tend to be a comparison of low and higher volume roads.

Armour (1984 b and c) used fatal crashes on New South Wales highways to produce relative crash rates by type of horizontal and vertical alignment and shoulder surface and these are shown in Table 7.2 and Table 7.3.

Table 7.2

Relative Crash Rates by Shoulder Type
and Horizontal Curvature

Horizontal Curvature	Shoulder Type Unsealed	Shoulder Type Sealed	All
Straight	1.0	0.3	0.7
Curve	5.9	1.5	3.5
All	1.8	0.6	

Source: Armour (1984 b and c)

Table 7.3

Relative Crash Rates by Shoulder Type and Grade

Grade	Shoulder Type Unsealed	Sealed	All
Flat	1.1	0.4	0.8
Grade	5.6	1.2	3.0
All	1.8	0.6	

Source: Armour (1984 b and c)

These results show that roads with sealed shoulders had lower crash rates regardless of horizontal and vertical alignment, but it was pointed out that no allowance had been made for the effect of volume. Also as sealing of shoulders is a recent practise it is likely that the roads with sealed shoulders may have been in slightly better conditions.

7.4 SHOULDER WIDTH

The effect of shoulder width was examined by Armour (1984 b and c) for unsealed shoulders, and shoulder widths of 1.1 to 2.0 metres indicated the lowest crash rates. However these results were based on low sample numbers.

Table 7.4

Relative Crash Rates for Unsealed Shoulders and Shoulder Width

Shoulder Width (Metres)	Crash Rates
0.1 - 1.0	3.9
1.1 - 2.0	1.3
2.1 - 3.0	2.4
GT 3.0	6.5
All	1.8

Source: Armour (1984 b and c)

The relative crash rates for shoulders of 1.1 to 2.0 metres were less than for any other class, but these results were based on a small sample.

McLean (1985) reviewed the results from a number of studies by converting them into an adjustment factor/roadway width values, with a base rate applying to a 13.2 metre roadway (two 3.6 metre lanes and two 3 metre shoulders). The results from developing countries showed higher rates perhaps because of the usage of shoulders by animal drawn carts etc. while the U.S. data was fairly consistent. Relative crash rates decreased as the roadway width tended towards the "ideal" width of 13.2 metres. The relative crash rates developed from U.S. studies were consistent and could be used to extend limited local data.

Charlesworth (1986) found from a survey of drivers stopped on road shoulders in Queensland that only 24% had done so because of an emergency and suggested that by the provision of turnouts the number of drivers stopped on shoulders could be reduced significantly thus reducing the need to provide wide shoulders.

7.5 SAFETY EFFECTIVENESS

There is strong evidence to indicate that the sealing of shoulders will produce crash reductions although there is some conflict within the research as to the desirable width of the sealed shoulder. Most researchers seem to agree that the provision of sealed shoulders are most cost-effective on grades and curves rather than the straight flat roads.

In Australia, Armour (1984 b and c) concluded that the crash rate for all rural roads with unsealed shoulders was 5.4 fatal crashes per 100 million vehicle kilometres and that for all rural roads with sealed shoulders was 1.8 crashes per 100 million vehicle kilometres. However, part of this difference may be due to the normal reduction in crash rates per hundred million vehicle kilometres as flows increase (RACV, 1985) since sealed shoulders are currently more likely to be used on busier roads.

7.6 COST-EFFECTIVENESS

In order to obtain some measure of cost-effectiveness the tables on relative effectiveness in Tables 7.2 and 7.3 have been factored to the estimates of the overall fatal crash rates of 5.4 and 1.8 fatal crashes per million vehicle kilometre made by Armour (1984 b and c) and then by the proportion of fatal to total rural crashes (refer Section 2.6) to obtain an estimate of total crashes per million vehicle kilometres).

Table 7.5

Crash Rates by Shoulder Type and Road Geometry (Crashes per Million Vehicle Kilometres)

Geometry	Shoulder Type		
	Unsealed	Sealed	All
Horizontal			
Straight	0.8	0.2	0.6
Curve	4.8	1.2	2.9
Vertical			
Flat	0.9	0.3	0.7
Grade	4.6	1.0	2.5
All	1.5	0.5	

Therefore in order to calculate an approximate benefit to cost ratio the following assumptions were necessary:

(a) the shoulders are constructed suitably for sealing to a width of 2 metres on both sides of the carriageway,

(b) the cost per kilometre including maintenance for 5 years is $25,000 for both shoulders, and

(c) the life of work is 5 years.

The resultant benefit to cost ratios are shown in Table 7.6.

Table 7.6

Approximate Benefit/Cost Ratio

Geometry	Annual Average Traffic				
	1000	2000	3000	4000	5000
Horizontal					
Straight	2	4	6	8	10
Curve	7	14	21	27	34
Vertical					
Flat	2	4	6	8	10
Grade	7	14	21	27	34

7.7 FURTHER RESEARCH

While there seems to be no disagreement between researchers as to the benefit of sealing the surface of shoulders there have been conflicting results presented on the worth of increasing the sealed width over 2.0 metres. Even the research of Armour (1984 b and c) indicated that wider sealed shoulders increased crash rates although it was emphasised that these results were based on small samples and that little confidence should be placed in the results. Charlesworth (1986) suggested that, if provisions were made to eliminate discretionary stops from shoulders of low volume roads, wide shoulders would not be necessary.

Therefore there seems to be considerable scope for further research to establish with more certainty the traffic volume and road geometric conditions which make shoulder sealing desirable and the optimum width for the travelled way (including sealed shoulders).

8.0 OVERTAKING LANES AND TURNOUTS

8.1 OVERTAKING LANES

8.1.1 Description

An overtaking (or passing) lane is defined as a third lane added in one or both directions of travel over part of a road length for the purpose of allowing slow-moving vehicles to be overtaken in situations where passing opportunities would otherwise be limited by sight distance or heavy opposing traffic volumes. Overtaking lanes are distinctly different from turnouts or passing bays which allow slow vehicles to pull aside and be overtaken (refer Section 8.3). Overtaking lanes are also referred to as auxiliary lanes and as truck climbing lanes in hilly or mountainous terrain.

On a two-lane road overtaking vehicles must overtake slower vehicles by entering the lane usually used by oncoming traffic. Therefore an overtaking opportunity requires a sufficiently large gap in the oncoming traffic for the overtaking manoeuvre, plus the distance travelled by that vehicle, plus a safety margin (Hoban, 1987). Moreover there must be adequate sight distance for the overtaking opportunity to be used. Whereas on high volume roads suitable gaps may be limited, sight distance restrictions on roads in rolling or mountainous terrain may limit overtaking opportunities irrespective of volume. It is the latter situation where overtaking lanes offer the greatest safety benefits.

Armour (1984a) found that overtaking is involved in about 10% of all rural casualty crashes. Research has demonstrated that on two-lane rural roads overtaking lanes improve overall traffic operations by breaking up traffic platoons and reducing delays caused by inadequate overtaking opportunities over substantial lengths of road. Whilst improved traffic operations is typically the primary objective, the installation of passing lanes has been found to usually reduce crash rates. Hoban (1987) believes that this reduction is a consequence of the wider seal width and the elimination of overtaking manoeuvres from the opposing traffic lane.

Guidelines for the provision of overtaking lanes have been developed by the National Association of Australian State Road Authorities (NAASRA) and overseas authorities. These include warrants (based on level of service), lane length, spacing between overtaking lanes, location and configuration. These have been detailed by Hoban (1987), and include:

- overtaking lane lengths (including tapers) ranging from a recommended 350 m for a 50 km/h design speed to 800 m for a 100 km/h design speed. Shorter overtaking lanes have been found to be more cost-effective;

- spacing, which is a function of traffic volumes and availability of overtaking opportunities, may be as low as 3 to 5 km.

8.1.2 Safety Effectiveness

Overtaking lanes can offer a substantial improvement in level of service (with a corresponding increase in road capacity), a reduction in delay, and a reduction in crash rates. Several studies have been undertaken to determine the degree of these improvements.

In the USA Harwood et al (1985) undertook a safety evaluation of 66 overtaking lanes and 10 short four-lane sections of road by analysing crash data extending over a one to five year period. The crash statistics at these treated sites were compared to sections of untreated road with similar characteristics.

Table 8.1 indicates that roads with passing lanes have lower crash rates than untreated roads. However, Harwood points out that none of the differences in Table 8.1 are statistically significant.

A matched pair-comparison of 13 of the passing lane sites and 13 corresponding untreated sites revealed a statistically significant reduction (at the 95% confidence level) of 38% and 29% for all crashes and fatal/injury crashes respectively. Applying these rates to Table 8.1, the mean crash rate reductions for passing lane sites with both directions combined would be 0.97 crashes/MVM for all crashes (reduced from 1.57) and 0.59 crashes/MVM for fatal/injury crashes (reduced from 0.83).

Because of their crash potential the transition (diverge and merge) areas of overtaking lanes have also been subjected to detailed analysis. The conclusion is that neither transition area presents a significant hazard, provided that a generous sight distance was provided for the merge area, especially on upgrades, and that the length and spacing of overtaking lanes is commensurate with the overtaking demand (Harwood et al, 1985; Homburger, 1986). Furthermore, Harwood et al found there were no other unusual safety problems associated with the passing lanes.

Table 8.1

Comparison of Crash Rates for Overtaking Lanes and Untreated Two-Lane Highways

		No. of Crashes		Mean Crash Rate[a] (Crashes/MVM)		
Type of Location	**No. of Sites**	**Total**	**Fatal & Injury**	**Exposure (MVM)**	**Total**	**Fatal & Injury**
Passing lane						
Treated direction[a]	66	305	133	271.0	1.13	0.49
Untreated direction	66	227	95	242.5	0.94	0.39
Both directions combined	66	532	228	513.5	1.04[b]	0.44[b]
Untreated two-lane highway (both directions combined)	13	430	226	273.5	1.57	0.83

Note: MVM = million vehicle miles

a Including lane-addition and lane-drop transition areas
b Based on average of crash rates for treated and untreated directions.

Source: Harwood et al (1985)

Hoban (1987) reviewed several other studies which demonstrated the safety effectiveness of passing lanes. Included was an extensive before/after study conducted by Rinde (1977) of sites widened to provide overtaking lanes in California and which found a reduction rate of 25 to 27%.

Harwood and Hoban (1987) combined the results of Harwood et al (1985) and Rinde (1977) to demonstrate a statistically significant reduction of 25% for roads in flat to rolling terrain (confidence level not reported).

It should be noted that two separate studies of the safety effectiveness of climbing lanes did not demonstrate significant reductions in crash rates. A study of a small number of climbing lanes in the US by Jorgensen (1966) could not identify any reduction, whereas in the UK Vorhees (1978) found a 13% reduction.

Harwood et al (1985) also investigated the safety effectiveness of short four-lane sections of road (to provide overtaking lanes in both directions). Crash data was collected for 9 such sites and compared to six untreated two-lane highway sections near all but one of the 9 treated sections. Crash rates at the treated sections were 34% and 43% lower for all crashes and fatal/injury crashes respectively, as illustrated in Table 8.2. However, these results were not statistically significant.

Table 8.2

Comparison of Crash Rates for Short Four-Lane Sections and Comparable Two-Lane Highways

		No. of Crashes			Mean Crash Rate[a] (Crashes/MVM)	
Type of Location	**No. of Sites**	**Total**	**Fatal & Injury**	**Exposure (MVM)**	**Total**	**Fatal & Injury**
Short four-lane section	9	106	69	89.6	1.18	0.77
Comparable two-lane highway	6	250	189	139.4	1.79	1.36

Source: Harwood et al (1985)

A matched-pair comparison of crash rates at 6 treated and 6 comparable untreated sections revealed lower rates - 53% and 52% for all crashes and fatal/injury crashes respectively. However, because of the limited sample size these results are also not statistically significant.

Harwood and Hoban (1987) prepared a summary of relative crash rates from recent research, which involved the above studies, as shown in Table 8.3. They concluded that a 25% reduction in all crashes can be achieved with overtaking lanes and a 35% reduction with a short four-lane section of highway. The reduction in fatal/injury rates is even higher.

Table 8.3

Relative Crash Rates for Improvement Alternatives

Alternatives	All Crashes	Fatal and Injury Crashes
Conventional two-lane highway	1.00	1.00
Overtaking lane section	0.75	0.70
Four-lane section	0.65	0.60

Source: Harwood and Hoban (1987)

8.1.3 Cost-Effectiveness

The conclusions of Harwood and Hoban as summarised in Table 8.3 can be applied to the extensive data reported in Table 8.1 to derive the following cost-effectiveness factors:

overtaking lane (single direction) - reduction of 0.25 total crashes per MVK.

four-lane section - reduction of 0.34 total crashes per MVK.

On the following assumptions:

a single treatment over a 7 km section of road (850 m long overtaking lane or 4-lane highway).

a 15-year lifetime.
AADT of 2000 vehicles,

the BCR for an overtaking lane is between approximately 3:1 and 6:1 and for a 4-lane highway between 2:1 and 4:1.

8.2 TURNOUTS / PASSING BAYS

8.2.1 Description

A turnout or passing bay is a short section of shoulder or added lane which allows slow vehicles to pull aside and be overtaken on a 2-lane rural road. The recommended length of either measure is typically 60-160 m and the width not less than 3.7 m (Harwood and Hoban, 1987). The spacing between turnouts is similar to overtaking lanes (refer Section 8.2).

A major difference between overtaking lanes and turnouts is that motorists are not encouraged to use the latter unless they are obviously holding up traffic.

Appropriate design and location are essential to the operational and safety effectiveness of turnouts. Even so, Harwood and St. John (1985) found that there was confusion on the part of some drivers as to how well designed turnouts were to be used. Moreover Hoban (1987) found that trucks rarely used turnouts, and they appeared to be most effective on recreational routes where tourist vehicles are generally prepared to suffer the slight inconvenience associated with pulling over.

8.2.2 Safety Effectiveness

A study of 42 turnouts in the US by Harwood and St. John (1985) found evidence of a 30% reduction in crash rates compared to nearby control sites.

Rooney (1976) studied 16 turnouts in California and also found a very low crash experience.

8.2.3 Cost-Effectiveness

On the limited data available the effectiveness of turnouts appear to be similar to that of overtaking lanes, although intuitively this does not seem logical. However the cost of turnouts is substantially less than overtaking lanes, and therefore the cost-effectiveness may be higher.

8.3 FURTHER RESEARCH

Additional research is warranted on turnouts because of their relatively low cost. However, driver behaviour in using turnouts must be studied to identify potential safety hazards associated with their use.

Research is also required to determine the use of all three options by Australian drivers with the objective of identifying the appropriate locations and design, and, if necessary, educating drivers as to their uses.

9.0 GEOMETRIC PARAMETERS

9.1 DESCRIPTION

The geometry of a road is concerned with its horizontal and vertical alignment. The geometric parameters define a road's path and location within the topography and land use which it traverses. These physical features in general act as constraints to a road's alignment.

Specifically, geometric parameters include:

- sight distance;
- curvature (horizontal and vertical);
- gradient;
- superelevation and crossfall.

Cross-sectional elements of road geometry (carriageway and lane widths, medians, etc.) are discussed in other sections of this Report.

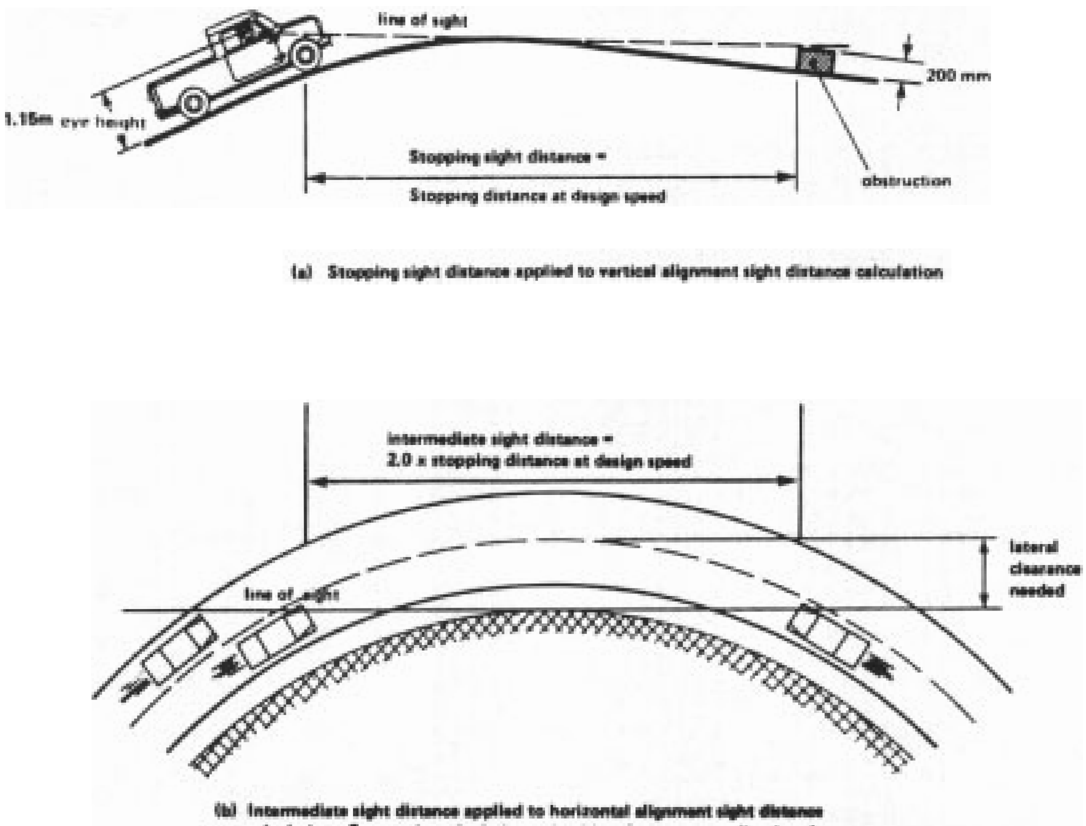
Sight distance is the distance at which a driver can see a specified object ahead of him. A principal aim in road design is to ensure that the driver is able to perceive any possible road hazards in sufficient time to take avoiding action, refer Figure 9.1. (Taken from Lay (1985) and NAASRA (1980).)

NAASRA (1980) describes:

> "The horizontal alignment of a road is usually a series of straights (tangents) and circular curves connected by transition curves."

> "The longitudinal profile of a road consists of a series of straight grades and vertical curves."

Superelevation is the transverse slope of the road on horizontal curves which is introduced to partially balance out the centrifugal force. The other balancing force is friction between the road surface and vehicle tyres.



(a) Stopping sight distance applied to vertical alignment sight distance calculation

(b) Intermediate sight distance applied to horizontal alignment sight distance calculation. For precise calculations, the drivers' eyes are usually placed on the centreline

Figure 9.1

Avoidance Sight Distances

Source: Lay (1985)

Crossfall is the transverse slope of the road on straight (tangent) sections which is in place to disperse surface water.

In order to produce a safe road design, sight distance, curvature and superelevation are related to the likely speed of operation of traffic on the road - its design speed.

9.2 OVERVIEW

The geometric conditions of a road are fundamental to its traffic capacity and (safe) operation.

Early roads were built with emphasis on capital cost and availability of materials and, in general, followed whatever extremes of topography were encountered.

As traffic volumes grew rapidly during the middle of this century, designs standards were developed for new and improved roads to provide adequate levels of service, minimum hazard and at reasonable cost.

However, these standards were developed through application of engineering principles and safety concepts rather than empirically from crash characteristics.

In the last thirty years, research has been progressively increasing on the correlation between the occurrence of crashes, the geometric features of roads, and traffic characteristics.

It is apparent that the majority of relevant research has been undertaken in the U.S.A., and certainly the U.S. literature has been more readily accessible to the consultants. However, useful research has been undertaken on N.S.W. roads by Cowl and Fairlie (1970), Cowl (1965), Donaldson (1974), and Boughton (1976). Also a general review of previous research was undertaken by Boughton (1975).

Other useful general literature reviews have been undertaken by the Texas Transportation Institute (1982) and Roy Jorgensen Associates (1978).

Most of the American research was based on the use of regression techniques to relate crashes to various design elements.

Some of the literature reviewed by the consultants in this study does not provide suitable information for an analysis of the cost effectiveness of geometric improvements.

It should be noted that geometric parameters are interrelated, e.g. sight distance and curvature.

In general terms, it is concluded that restrictions on sight distance and curvature, and the presence of grades lead to increased crash rates. Adequate superelevation and crossfall are required to inhibit hydroplaning on wet pavements.

9.3 SIGHT DISTANCE

9.3.1 Review (of Safety Effectiveness)

McBean (1982) found that on rural roads sight distances shorter than 200 metres are relatively more likely to be found at crash sites. However, he intimated that this was due to the association between sight distance and curvature, i.e. that bends tend to reduce sight distance.

The number of crashes and control sites relating to sight distances measured in five ranges are shown in Table 9.1.

This study is significant because it attempted to compare sites with similar characteristics except for the variable being assessed. Note there is no identification of road type.

Neuman and Glennon (1983) attempted to systematically evaluate the cost-effectiveness of spot improvements of stopping sight distance (SSD) - deficient locations. They found that SSD restrictions did contribute to crash risk. They developed a matrix of crash rate reduction factors to describe the hypothesised relations between crash rate and SSD conditions, as reproduced in Table 9.2 below.

Table 9.1

Sight Distance at Crash and Control Sites

Control Sites	Crash Sites 0-100m	101-199m	200-299m	300-399m	GE 400m	Totals
0-100m	10	13	5	2	6	36
101-199m	13	18	5	5	9	50
200-299m	7	12	11	4	6	40
300-399m	3	12	1	1	3	20
GT 400m	10	22	7	4	8	51
TOTALS	43	77	29	16	32	197

Source: McBean (1982)

Table 9.2

Hypothesised Crash Rate Factors(1) for Evaluation of SSD Restrictions

Character of Geometric Condition within SSD Restriction	Severity of SSD Restriction by Vehicle Speed(2) 0 km/hr	15 km/hr	25 km/hr	35 km/hr
Minor hazard	0	0.5	1.2	2.0
Significant hazard	0.4	1.1	2.0	3.0
Major hazard	1.0	1.8	2.8	4.0

(1) Factor multiplied by average crash rate is crash rate attributable to the combined effects of the roadway geometry and SSD restriction.

(2) Increment of speed under highway operating speed for which SSD is sufficient, i.e. differential between design speed and operating speed.

Source: Neuman & Glennon (1983)

Other studies have been inconclusive on the safety effects of SSD. Schoppert (1957) concluded that although sight distances restriction is related to crashes, it does not serve as a good predictor of the number of crashes.

Raff (1953) also found some significance between frequency of sight distance restrictions and crashes as set out in Table 9.3.

Table 9.3

Crash Rates on Two-Lane Tangents by Frequency of Sight-Distance Restrictions

Frequency of Restrictions No. Per Mile	(All States Without Adjustment) Number	Per Mill. Vehic.-Miles
0 - 0.9	3,472	2.0
1.0 - 1.9	1,061	2.5
2.0 - 2.9	891	3.1
3.0 - 3.9	684	3.0
4.0 - 4.9	354	3.0
5.0 - 5.9	12	2.7

Note: The author indicates that the crash rates are statistically significant
Source: Raff (1953)

Gupta and Jain (1975) and Sparks (1968) did not reach any conclusion on sight distances.

A number of researchers did not analyse sight distance separately from curvature because they are inter-related.

As can be seen, there is a dearth of statistical data relating crash rates to sight distance. However, it is concluded that restrictions to sight distance do contribute to road crashes.

9.3.2 Cost-Effectiveness of Improving Sight Distance Restrictions

Neuman and Glennon (1983) undertook a most pertinent study into the cost-effectiveness of improvements to stopping-sight distance (SSD) safety problems on roads of different classification in the U.S.A. They established a framework for classifying the crash potential of such locations and determining the hypothetical safety benefit of treatments.

It has not been possible, within the scope of this study, to translate every component of their analysis to Australian conditions, especially the hypothetical crash reduction factors, and the relationship between crash rate and traffic volume. Therefore, the U.S. cost-effectiveness analysis is presented. It is considered that this will provide a reasonable indication of the magnitude of benefits that could be derived from similar measures in Australia. However, for each measure selected from presentation, the expected reduction in the (hypothesised) crash rate is given, which can then be compared with the cost of implementation, similarly to other safety improvements outlined in this report.

Annual number of crashes attributable to the SSD restriction was calculated using the formula:

$$\text{Crashes} = 365 \times ADT \times RH \times LR \times FAR \times 10^{-6}$$

where

ADT = average daily traffic volume

RH = average crash rate

= 1.5/MVKm for rural 2-lane road

= 0.5/MVKm for rural freeway

= 5.3/MVKm for urban arterial

= 1.1/MVKm for urban freeway

LR = length of SSD restriction given in Figure 9.2

FAR = hypothesised crash rate factors given in Table 9.2.

An average cost per crash (for all crashes) was taken as US$11,048 for rural 2-lane highways.

Typical construction costs of a range of countermeasures were given as reproduced in Table 9.4.

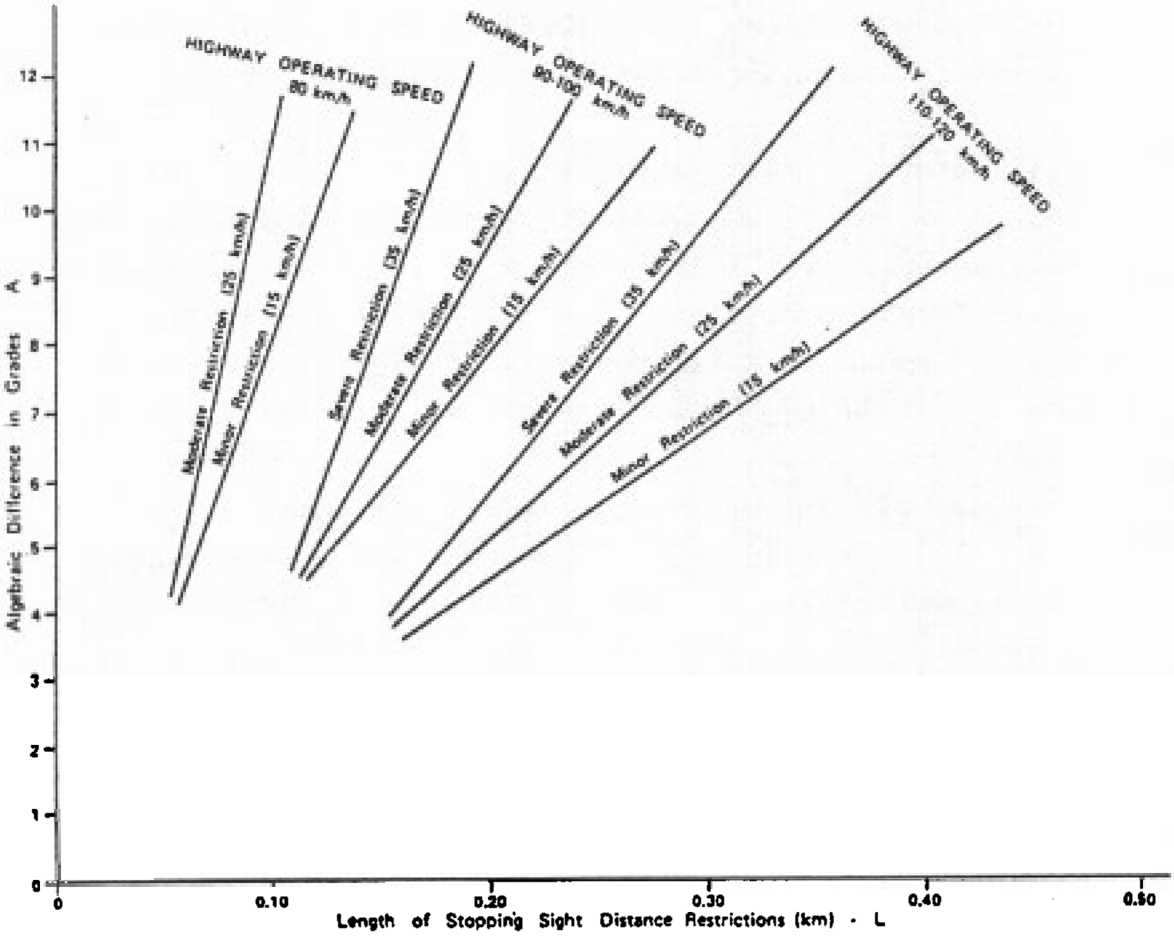


Figure 9.2

Relations Among Grades, Severity,
and Length of SSD Restrictions on Vertical Curves

Source: Neuman and Glennon (1983)

Costs of implementation were discounted to an annual present worth amount assuming a 20 year life for pavements and 40 year life for earthworks and roadbase, etc. A benefit/cost ratio of 1.0 or greater indicated potential cost-effectiveness.

Table 9.4

Cost of Construction and Implementation for SSD Countermeasures on Two-Lane Rural Highways

Countermeasures	Cost of Construction & Implementation (US$)
Lengthen vertical curve	
Low A (1)	
15 km/h increase in SSD	120,000-170,000
35 km/h increase in SSD	125,000-185,000
Moderate A	
15 km/h increase in SSD	185,000-270,000
35 km/h increase in SSD	200,000-300,000
High A	
15 km/h increase in SSD	270,000-320,000
35 km/h increase in SSD	300,000-350,000
Clear trees or brush from inside horizontal curve	
15 km/h increase in SSD	3,000-6,000
35 km/h increase in SSD	9,500-15,000
Flatten horizontal curve within SSD restriction	150,000-200,000
Move intersection on Y-diverge away from SSD restriction	120,000-220,000
Widen roadway at crest vertical curve	70,000
Widen narrow structure	(2)

(1) A = algebraic difference in grades

(2) Generally prohibitive

Source: Neuman & Glennon (1983)

The results of the Neuman and Glennon analysis for a range of countermeasures on rural 2-lane highways is reproduced as Table 9.5.

Using the above approach, the consultants calculated the approximate benefit/cost ratio for particular examples of countermeasure, as shown in Table 9.6.

Table 9.5

Cost-Effectiveness of Spot-Improvement Countermeasures to SSD Safety Problems on Rural Two-Lane Highways

Countermeasures	ADT Level Required for Potential Cost-Effectiveness	Geometric, Operational, and Traffic Conditions
Lengthen vertical curve with a 25 km/h deficiency in SSD	13,000-15,000 17,000-19,000	Significant hazard within SSD restriction Minor hazard within SSD restriction
Lengthen vertical curve with a 35 km/h deficiency in SSD	10,000-14,000 13,000-17,000	Significant hazard within SSD restriction Minor hazard within SSD restriction
Flatten sharp horizontal curve with a SSD deficiency	11,000-12,000	Design speed of curve at least 25 km/h lower than design speed of highway
Clear trees or minor obstructions from inside of horizontal curves	200-400	At least 25 km/h SSD deficiency
Cut earth from inside of horizontal curve	500-1,000	At least 25 km/h SSD deficiency
Move intersection or Y-diverge away from SSD restriction	NC-E	
Widen roadway at very short crest vertical curve	NC-E	Very narrow, low-volume minor roads
Widen narrow bridge within SSD deficiency	NC-E	
Warning signs, delineation, and pavement-marking schemes	Any level	Minor hazard within 25 to 35 km/h SSD restriction; significant or major hazard within 15 to 35 km/h SSD restriction

Note: NC-E = Not cost-effective under typical geometric, operational, or traffic conditions
Source: Neuman & Glennon (1983).

Table 9.6

Calculation of Benefit/Cost Ratios of SSD Countermeasures

SSD Restriction (km/h)	15	15	15	35	35
Geometric hazard condition (1)	Minor	Signif.	Signif.	Signif.	Signif.
"A", difference in grades	5	5	8	5	8
Operating speed (kph)	90-100	90-100	90-100	90-100	90-100
ADT	20000	15000	15000	15000	15000
Implementation cost (US$)	120000	120000	185000	185000	300000
Annual PV factor	0.125	0.125	0.125	0.125	0.125
RH	1.50	1.50	1.50	1.50	1.50
LR	0.13	0.13	0.20	0.11	0.15
FAR	0.5	1.1	1.1	3.0	3.0
No. annual attributable crashes	0.711	1.174	1.807	2.710	3.696
Average cost/crash (US$)	11048	11048	11048	11048	11048
Annual crash benefit (US$)	7863	12975	19961	29941	40829
Annual PV costs (US$)	15000	15000	23125	23125	37500
Benefit/cost ratio	0.52	0.87	0.86	1.29	1.09
Cost-effective?	No	No	No	Yes	Yes

(1) A compounding geometric condition, e.g. close intersection.

Safety benefits achieved by increasing SSD alone (e.g. by lengthening or flattening vertical curves) are determined by reading laterally from right to left in Table 9.2. For example, for the removal of a minor SSD restriction of 15 km/h Neuman and Glennon's method hypothesises an crash rate reduction from 1.5 to 1.0, i.e. 33%. Similarly, removal of a major SSD of 35 km/h a reduction of 3.0 to 1.0 or 67% would be hypothesised.

Olsen, Cleveland, Fancher, Kostyniuk and Schneider (1984) undertook a study of the parameters affecting stopping sight distance. Part of this study involved an investigation of SSD on safety. It was a controlled test where 10 sites with below standard SSD were compared with control sites of the same characteristics, except for the length of SSD which was in excess of 210 metres in 9 cases and 163 metres in a single case, i.e. above the AASHTO standard. However, the measure of safety was the crash count only and SSD was limited at the test sites because of the presence of a vertical curve. No differentiation was made of crash type.

The analysis showed a statistically significant difference at the 95% confidence level between the two crash samples. The sites meeting the AASHTO 1965 standard experienced 34% fewer crashes during the study period than did sites with limited available SSD. The average SSD of the test sites was only 71 metres which is sufficient for a design speed of only about 60 km/h under NAASRA rural road standards.

9.4 HORIZONTAL AND VERTICAL ALIGNMENT (Including Gradient)

9.4.1 Review (of Safety Effectiveness)

These parameters are treated together because some researchers have considered combined horizontal and vertical curve sections of road, and gradient is clearly an integral part of vertical geometry.

Boughton (1976) undertook an analysis of fatal crashes occurring on N.S.W. rural, 2-lane, 2-way State highways with gravel shoulders, based on detailed crash statistics from N.S.W. in the period 1969 to 1971 inclusive.

Type and number of crash by horizontal and vertical alignment are shown, respectively, in Tables 9.7 and 9.8. Crashes by combined horizontal and vertical alignment are shown in Table 9.9.

Table 9.7

Type of Fatal Crash by Horizontal Alignment[1]

Type of Crash	Straight	Radius of Curvature (m)[1] LE 120	180	240	300	460	610	910	1220	1520	GT 1520	NK[2] Radius	NK[2]	Total
Single-vehicle														
Ran off road	61	15	17	16	26	21	23	9	2	1	1	11	1	204
Over turned	18	7	2	4	3	3	6	3	1	1	-	2	-	50
Out of control	11	2	5	1	4	2	4	-	-	1	-	4	-	34
Sub-Total	90	24	24	21	33	26	33	12	3	3	1	17	1	288
Multi-vehicle														
Head-on and ss opp.[3]	90	26	15	18	20	23	14	8	3	1	2	14	1	235
Rear end and ss same[3]	24	-	-	-	-	1	4	-	-	-	-	-	-	29
Angle	11	1	-	1	-	2	2	-	-	-	-	1	-	18
Sub-Total	125	27	15	19	20	26	20	8	3	1	2	15	1	282
All Crashes[4]	219	52	41	40	53	52	53	20	6	4	3	32	2	577

(1) Conversion factor of 1 ft = 0.3048 m has been used, figures rounded to nearest 10 m and where, for example, 300 m stands for the interval 240-300.
(2) NK = Not known
(3) ss opp. = sideswipe opposite direction
ss same = sideswipe same direction
(4) Includes "other" crashes

Source: Boughton (1976)

Table 9.8

Type of Fatal Crash by Vertical Alignment

Type of Crash	Vertical Alignment — Gradient (percent) -10	-9	-8	-7	-6	-5	-4	-3	-2	-1	0	1	2	3	4	5	6	7	8	9	10	Sag	Crest	Level	NK	Total
Single-vehicle																										
Ran off road	-	-	-	3	7	9	11	12	12	8	5	8	6	14	3	5	-	-	-	-	-	3	6	79	13	204
Overturned	-	-	2	1	3	4	1	3	2	4	-	3	2	1	-	1	-	-	-	-	-	2	3	17	1	50
Out of control	-	1	1	-	5	1	-	1	3	2	-	1	1	1	2	-	1	-	-	-	-	1	2	11	-	34
Sub-Total	-	1	3	4	15	14	12	16	17	14	5	12	9	16	5	6	1	-	-	-	-	6	11	107	14	228
Multi-vehicle																										
Head-on and ss opp.(1)	-	1	2	3	3	10	8	11	18	12	2	9	14	15	12	5	2	7	1	-	1	7	13	71	8	235
Rear end and ss same(1)	-	-	-	-	1	1	-	2	2	2	-	1	1	3	1	-	-	-	-	-	-	-	-	14	1	29
Angle	-	-	1	-	-	-	-	-	4	-	1	-	1	-	-	1	-	-	-	-	-	-	-	9	-	18
Sub-Total	-	1	3	3	4	11	8	13	24	14	3	10	16	18	13	6	2	7	1	-	1	7	14	94	9	282
All Crashes(2)	-	2	6	8	19	25	20	31	41	28	9	23	25	34	18	12	3	7	1	-	1	13	25	203	23	577

(1) ss opp. = sideswipe opposite direction
ss same = sideswipe same direction

(2) Includes "other" crashes

Source: Boughton (1976)

Table 9.9

Fatal Crashes by Combined Horizontal and Vertical Alignment[1]

Horizontal Alignment	Vertical Alignment										
	Gradient (percent)										
	Level	0-1.9	2-3.9	4-5.9	6-7.9	GE 8	NK	Sag	Crest	Not Known	Total
Straight	101	31	40	20	10	1	6	4	6	-	219
Curvature (m)											
LE 120	10	-	18	9	6	1	3	1	-	4	52
180	9	2	11	8	5	1	-	3	1	1	41
240	12	1	11	6	5	2	-	-	1	2	40
300	16	3	10	15	5	-	-	2	1	1	53
460	12	9	18	7	1	1	-	1	3	-	52
610	22	6	12	5	4	1	-	-	3	-	53
910	7	6	3	1	-	-	-	1	1	1	20
1220	2	3	-	1	-	-	-	-	-	-	6
1520	1	-	3	-	-	-	-	-	-	-	4
GT 1520	1	-	1	-	-	-	-	-	1	-	3
NK	10	2	4	2	1	1	2	1	8	1	32
Not known	-	-	-	-	-	-	2	-	-	-	2
Total[1]	203	63	131	74	37	8	13	13	25	10	577

(1) The totals in this table do not correspond exactly with Table 9.8 because Table 9.8 was compiled by rounding

Source: Boughton (1976).

Of those crashes which occurred on a horizontal curve of known radius (62% of total), approximately 90% of both single-vehicle crashes (161 out of 180) and multi-vehicle crashes (127 out of 141) were associated with curves of radius not greater than 610 metres. This corresponds to the figures of 600 metres found in a number of American studies (Highway Users Federation for Safety Mobility (1971), Raff (1953)).

44% of the crashes were known to have occurred on grades, including 2% on sags and 5% on crests.

40% of the crashes (216 out of 577) occurred on a combined horizontal curve and vertical grade. 33% (72) of these occurred on curves of radius less than 460 m in conjunction with gradients greater than 4%. Previous studies (see below) have highlighted the problem of the combination of curves of radius less than 450 m and gradients above 4%.

Crash rates were not determined in the above study.

Cowl and Fairlie (1970) from an analysis of fatal crashes on N.S.W. rural State highways between 1966 and 1968, inclusive, found that 70% of fatal crashes on horizontal curves occurred on curves of 300 m or less, and 82% (294 out of 360) on curves of 450 m or less.

In an analysis of all crashes occurring on the Sydney/Newcastle freeway, Donaldson (1974) produced crash rates of different horizontal curve radii - refer Table 9.10. This showed a significantly higher rate for curves less than 450 m.

With regard to vertical alignment, Donaldson found no relationship with crash frequency. However, of the ten 370 metre horizontal curves which exhibited very high crash rates six occurred in conjunction with 6% grades.

Raff (1953) found that there was a direct relation between curvature and crash rate on all types of road. The sharper the curve the higher the crash rate - see Table 9.11. He also found that, in general, crash rates were higher when the frequency of curves per kilometre increases, except for sharp curves less than 175 m.

Table 9.10

Crashes Occurring on Curves - Sydney/Newcastle Tollway New South Wales 12/12/68 to 31/12/73

Curve Radius (x metres)			Number of Curves L	R	T	Crashes L	R	T	Crash Rate L	R	T
	X	LE 450	5	5	10	65	70	135	13.0	14.0	(1) 13.5
45	-	610	7	7	14	52	36	88	7.4	5.1	6.3
611	-	760	3	5	8	18	22	40	6.0	4.1	5.0
761	-	920	3	3	6	1	5	6	0.3	1.7	1.0
921	-	1220	7	7	14	19	30	49	2.7	4.3	3.5
1221	-	1530	1	-	1	6	-	6	6.0	-	6.0
1531	-	1830	3	4	7	20	6	26	6.7	1.5	3.7
	X	GT 1830	2	3	5	3	2	5	1.5	0.7	1.0
Total			32	34	66	184	171	355	5.8	5.0	5.4

L = curve left
R = curve right
T = all curves = L + R

Note that all curves in the range X LE 450 m were 370 m curves

(1) Equals 2.5 times average rate

Source: Donaldson (1974)

In a study of 25500 crashes on 2100 km of expressway for the years 1953 to 1955, Bitzel (1956) showed that there was a significant increase in crash rate for grades above 4% when in combination with horizontal curves, refer Table 9.12.

He also showed clearly that crash rates increased with increasing grade, refer Table 9.13.

Table 9.11

Crash Rate Versus Curvature

	Accident Rates (All States, Without Adjustment) (1)									
	Two-Lane Roads		Three-Lane Roads		Four-Lane Roads					
					Undivided		Divided		Controlled Access	
Radius(2) m	umber	Per Mil. Vehicle-Kms	lumber	Per Mil. Vehicle-Kms	umber	Per Mil. Vehicle-Kms	Number	Per Mil. Vehicle-Kms	Number	Per Mil. Vehicle-Kms
> 600	504	1.0	11	1.1	98	1.2	95	1.1	180	1.0
300-600	596	1.6	11	1.8	90	1.6	65	1.5	162	1.4
175-300	338	1.8	6	2.2	16	2.1	5	1.9	38	2.8
< 175	354	2.2	11	4.6	3	0.8	12	4.2	0	-
Tangents	474	1.4	227	1.6	1348	1.7	982	1.8	774	1.1

(2) Approximately converted from "degree of curvature".

(1) Approximately converted from MVM.

Source : Bitzel (1956)

Table 9.12

Crash Rates Related to Horizontal Curvature and Gradient for German Expressways

Curve Radius (m)	Crashes Per Million Veh. km on Gradients (in percent) 0 - 1.99	2 - 3.99	4 - 5.99	6 - 8.0
GT 4000	0.28	0.20	1.05	1.32
300 - 4000	0.42	0.25	1.30	1.55
200 - 300	0.40	0.20	1.50	1.70
100 - 200	0.50	0.71	1.86	2.01
LT 100	0.73	1.06	1.93	2.35

Source: Bitzel (1956)

Table 9.13

Crash Rates Related to Grades on German Expressways

Roadway Grade %	Crash Rate Per MVKm
0 - 1.9	0.46
2 - 3.9	0.67
4 - 5.9	1.90
6 - 8.0	2.10

Source: Bitzel (1956)

In an extensive study of crash rates as related to design elements of all types of rural highways, Kihlberg and Tharp (1968) found that the presence of gradients, curves, intersections and structures increases crash rates. This effect was most marked for intersections and least marked for gradients. Simultaneous presence of several of these factors generates crash rates typically two or three times higher than rates on highway segments free of such interfering factors. There is some indication that the multi-vehicle crash rate is specifically associated with the presence of intersections, and the one-vehicle crash rate with the presence of curvature or structures. Severity rates generally are not affected by the geometric features.

Partitioning of grading and curvature by magnitude, above 4% and below about 435 metres radius, respectively, showed generally no change in the effect on crash rates. However, in certain States with two-lane highways significant increases in crash rates were found for these two conditions.

Regression analysis was utilised to form predictive equations relating crashes to the various design parameters.

Neuman, Glennon and Saag (1983) through regression techniques relate various design features of 3557 sites from four States of the U.S.A. to the probability of a curve being a high crash location, as shown in Table 9.14.

Table 9.14
Probability that a Highway Curve Site is a High Crash Location

Low Roadside Hazard (20)
High Pavement Skid Resistance (50)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Long (.30)	0	50	53	58	-	-
	4	37	39	45	-	-
	8	22	24	27	-	-
Moderate (.17)	0	42	45	50	-	-
	4	30	32	37	-	-
	8	18	20	23	-	-
Short (.05)	0	34	37	42	52	64
	4	23	25	30	38	52
	8	14	16	19	26	38

Moderate Roadside Hazard (35)
High Pavement Skid Resistance (50)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Long (.30)	0	86	87	88	-	-
	4	76	77	80	-	-
	8	66	67	70	-	-
Moderate (.17)	0	82	83	85	-	-
	4	69	71	74	-	-
	8	58	60	62	-	-

Cont'd.

<u>Table 9.14</u>
Cont'd.

High Roadside Hazard (50)
Moderate Pavement Skid Resistance (35)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Short (.05)	0	74	76	78	85	91
	4	62	64	66	77	86
	8	50	52	54	65	79
Long (0.30)	0	91	92	93	-	-
	4	85	89	90	-	-
	8	73	79	82	-	-
Moderate (.17)	0	87	89	90	-	-
	4	78	84	86	-	-
	8	66	72	75	-	-
Short (.05)	0	82	84	86	90	94
	4	71	76	79	84	89
	8	59	65	68	74	82

High Roadside Hazard (50)
Low Pavement Skid Resistance (20)

Curve Length (mi.)	Shoulder Width (ft.)	Degree of Curve 1	3	6	12	20
Long (.30)	0	97	97	98	-	-
	4	95	95	96	-	-
	8	92	92	93	-	-
Moderate (.17)						
Short (.05)	0	96	97	97	98	99
	4	94	95	95	96	97
	8	91	92	92	93	94

Tabulated values are percent probabilities as given by Equation 1 and Figure 1 (ref. article).

Source: Neuman, Glennon and Saag (1983)

The method indicates that improving roadside design, pavement skid resistance, and shoulder width may be valid countermeasures.

McBean (1982) also found that radii less than about 450 metres were more likely to be found at crash sites for both single and multiple vehicle crashes. He found no evidence that the sense of the bend (right or left hand) has any influence on crash risk for curves less than 450 metres. However, his data revealed no evidence of a relationship between gradient and crash risk.

Dunlap et al (1978) studied crash records for the Pennsylvania and Ohio Turnpikes, and analysed the effects of horizontal and vertical alignment on crash rates. On the Ohio Turnpike, they found no significant crash dependence on either grade or curvature, except that a 1° curve (1750 m radius) on a 3% downgrade had a very high crash rate. The data showed the Pennsylvania Turnpike crash rate was not dependent on grade, but it did increase with increasing curvature.

In the study of the economics of design standards for low-volume rural roads in the U.S.A., Oglesby and Altenhofen (1969) found that:

> "In contrast to upgrading the cross section, which calls for continuous reconstruction along the length of the road, there is indication that spot improvements to flatten sharp horizontal curves may in many situations pay their way, primarily because the costs of slowing and accelerating are eliminated. Also, the evidence is strong that accidents occur at occasional sharp curves in relatively straight alinements; this offers an additional argument for curve flattening. Similar advantages do not seem to accrue where vertical alinement is improved, since drivers do not slow down nor are there many accidents where vertical sight distance is impaired."

There is strong evidence to suggest that the following geometric conditions result in increased crash rates on all rural roads:

(i) reducing radius (or increasing curvature), especially below values of about 450 to 600 metres;

(ii) combinations of horizontal and vertical alignment, particularly where combined conditions are more than moderate.

There is weaker evidence to suggest that gradients alone are a contributing cause to increased crash rates on rural roads.

9.4.2 Cost-Effectiveness of Improving Curvature and Grade Restrictions

In Boughton (1976) fatal crash numbers only are reported, with no real indication of either rate or exposure. Therefore, it is perhaps inappropriate to estimate potential cost-effectiveness values from such data.

Potential reduction in fatal crash numbers can be broadly determined as follows:

(i) By eliminating all horizontal curves equal or less than 610 metres could eliminate up to 50% of existing fatal crashes.

(ii) By eliminating all combined horizontal/vertical curves with curvature less than 460 metres and gradients greater than 4% could eliminate up to 33% of existing fatal crashes.

The elimination of curves less than 610 metres radii, in low cut and fill situation, typically requires complete reconstruction costs between \$120,000 and \$180,000 per kilometre. Combined horizontal curves less than 460 metre radii with grades greater than 4% require complete reconstruction with larger cut and fill sections of roadway. Typically costs would range from \$200,000 to \$350,000 per kilometre.

The above cursory estimates should be taken as absolute maxima, as no allowance has been made for likely increased crashes of other categories (on curves greater than 610 metres and straights, in the first estimate) following hypothetical elimination of subject categories.

Using the data in Table 9.11 (Raff, 1953), the approximate order of crash savings by eliminating horizontal curves less than 300 metres and 600 metres for different road types is given below (Table 9.15). This analysis uses the crash cost and average severity index obtained from South Australian data (cost of PDO = \$2600, SI = 16.7) to obtain a cost saving.

Table 9.15

Potential Cost Savings

Elimination of Curves of	Crash Rate (MVKm)	Reduction (%)	Unit Potential Cost Saving ($)
Two-Lane Roads			
LT 300 m	0.7	35	30400
LT 600 m	0.8	45	34700
Three-Lane Roads			
LT 300 m	2.0	58	86800
LT 600 m	1.8	62	78200
Four-Lane Roads			
Undivided			
LT 300 m	Nil	Nil	Nil
LT 600 m	0.3	19	13000
Divided			
LT 300 m	1.8	57	78200
LT 600 m	1.4	56	60800
Controlled Access			
LT 300 m	1.6	57	69500
LT 600 m	1.1	53	47800

(Note: The above crash rate reduction factors were calculated from the original table not Table 9.11, in order to reduce compounding rounding errors in converting to MVKm.)

Thus at a flow of 5000 AADT, straightening of a 300 m radius bend of 300 m length would give a NPB of $128,900 for a cost of $62,000, a BCR of 2:1.

Leeming (1968) produced a forecast of crash reductions for various road engineering works, as reproduced in Table 9.16 below. No information was to hand as to the extent of improvements, but the results showed significant crash reductions.

9.5 SUPERELEVATION AND CROSSFALL

9.5.1 Review (of Safety Effectiveness)

There is a limited amount of literature which considers either superelevation separately from horizontal curvature, or crossfall.

Dart and Mann (1970) concluded that the pavement crossfall has an important effect on the crash rates as related to traffic volumes. They found that the crash rate increases as crossfall decreases - see Figure 9.3.

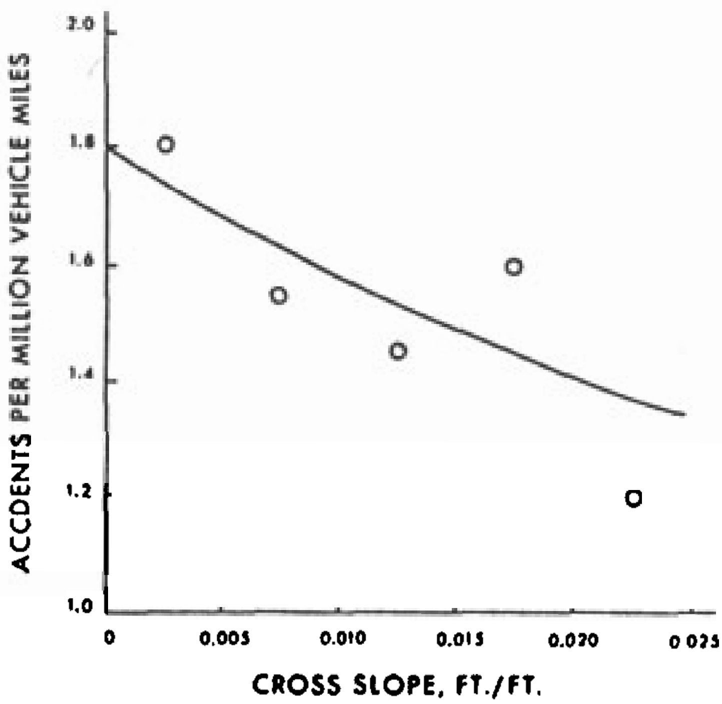


Figure 9.3

Crash Rate Versus Pavement Cross Slope (Crossfall)

Source: Dart and Mann (1970)

Table 9.16

Forecast of Crash Reductions

Line	No. of Counties	No. of sites in sample	Type of change	Ratio of accidents after the change to those before: All injury accidents – Ratio	All injury accidents – Significance	Fatal, serious combined – Ratio	Fatal, serious combined – Significance	Fatal only – Ratio	Fatal only – Significance	Total fatal accidents at all sites		Changes in accidents all sites all inj. accidents: +	No change	−
(a) Civil engineering works														
1	4	5	Motorways (both roads after)	0.81	Yes 5%	0.72	Yes 5%	0.93	No	98	83	—	1	4
2	9	19	By-passes to towns („ „)	0.67	Yes 1%	0.88	No	0.94	No	23	26	3	4	12
3	18	39	Dual carriageways	0.68	Yes 5%	0.78	No	0.75	No	47	33	7	8	24
4	15	37	Widening two lanes to three	0.78	Nearly 10%	0.90	No	0.52	Yes 5%	32	17	7	10	20
5	19	53	Realignment of two-lane roads	0.31	Highly 0.1%	0.32	Highly 0.1%	0.25	Yes 5%	13	3	3	13	37
6	17	22	Roundabouts	0.42	Yes 5%	0.44	Yes 5%			8	2	1	4	7
7	7	16	Stagger crossroads	0.28	Highly 0.1%	0.19	Highly 0.1%			8	—	—	4	12
8	10	16	Junction improvements	0.34	Highly 0.1%	0.46	Very nearly 5%			9	2	—	6	10
9	7	28	Miscellaneous minor improvements	0.55	Highly 0.1%					9	1	2	14	12
10	6	31	Surface dressing bends	0.13	Highly 0.1%	0.21	Yes 1%			7	1	—	3	28

Source: Leeming (23)

The Highway Users Federation for Safety and Mobility HUSFM7 report (1971) in a study based on 6000 crashes of rural highways in Louisanna considered the relative contributions of geometric features (lane and shoulder widths, crossfall, horizontal and vertical alignment, and obstructions) to crashes. It found that low pavement crossfall had the most important effect. However, only 41% of the variation in crashes rates were explained by the geometric factors used in the study.

(Note, the above two references were not sighted and the data are not available for analysis of cost-effectiveness.)

Dunlap et al (1973) studied in detail the causes of crashes on the Ohio Turnpike. Of the total crashes, 67% involved skidding during wet weather. However, the curve in question only had a superelevation of 0.0156 m/m below the AASHTO standard. They recommended that larger superelevations be used on large radius curves. In the above example they estimated that increasing the superelevation to 0.06 would reduce water depths by about one third and thus reduce wet weather crash rates.

Transition curves are predominantly used to connect tangents to relatively small radius curves as a means of gradually introducing curvature and superelevation. No crash studies have been reported on the application of transition curves. However, a study by Segal and Ranney (1980) analysed the vehicle dynamics for transition curves. They used the computer simulation model, Highway-Vehicle-Object Simulation Model (HVOSM) for the analysis of three low to moderate radii curves. They found that vehicles' lateral acceleration with no transition curve was as much as 50% greater than the steady-state acceleration, while the spiral transition simulated less than 10%. They concluded no transition was the worst case, compound transitions better, and the spiral allowed the easiest path to follow.

McBean (1982) omitted superelevation from his analysis because it was generally associated with the presence of a horizontal curve. Analysis of crossfall showed no apparent relationship with crash risk.

It can be concluded that road safety is improved if adequate crossfall and superelevation are maintained.

9.5.2 Cost-Effectiveness of Improving Superelevation and Crossfall Limitations

Dart and Mann (1970) is the only reference sighted which allows an assessment of cost-effectiveness of improvements to crossfall (in general).

From Figure 9.3, increasing crossfall would produce approximate reduction in crash rates (per million vehicle kilometres) as follows:

Crossfall Increase (m/m)	Crash Rate (MVKm)	Reduction %
0.005 to 0.010	0.06	6
0.010 to 0.015	0.06	6
0.015 to 0.020	0.05	5

For 2-lane rural highways the cost to increase crossfall by 0.005 m/m would be $40-50,000/km. However, this measure would probably only be contemplated if a clear deficiency existed or there was a high crash rate. The cost-effectiveness is therefore considered to be low. In combination with curvature however, the resulting "resheet" provided, could improve frictional characteristics.

Large changes in crossfall cannot be achieved by re-sheeting and re-construction of the base-course would be needed. The cost of this improvement could be between $70-80,000/km.

9.6 CONCLUSIONS AND FURTHER RESEARCH

The analysis of the safety effectiveness of geometric parameters and conditions has considered rural roads only, although all road types within the category - 2-lane, 3-lane, 4-lane, divided, undivided - were included where literature was available.

There is a substantial body of past research undertaken in this field, the majority from the U.S.A. Some important papers from Australia were also reviewed, but few were readily obtainable from Europe in general and the U.K. particularly.

It is interesting to note that of the 25 papers and reports reviewed that the average date of publication was 1973.

It is clear from the review of literature and analysis of safety effectiveness that there is considerable evidence to support the following findings:

(a) Sight distance restrictions contribute to higher crash rates.

Measures to improve restrictions can be cost-effective, but could require relatively high traffic volumes to warrant.

It is considered by the consultants that these treatments are applicable to undivided roads only.

The analysis was based mainly on American research which was difficult to translate to Australian traffic crash conditions. Not all research reviewed was conclusive, particularly the main source of Neuman and Glennon (1983) which was based on hypothetical crash reduction rates.

It is possible to reassess the BCR's given in Table 9.6 for Australian conditions and costs. Taking the central column as an example, the implementation cost is estimated at \$200,000 and the average crash cost as \$43,400. Taking a 10 year life gives a NBP of \$286,400. The NPC is estimated as \$224,000. This gives a BCR of 1.3 compared to the 0.9 estimated from the U.S. figures. This is however at an AADT of 15,000 vehicles and the BCR would thus only be above one for flows of about 10,000 and above.

It is considered that new Australian research would be most worthwhile in this area.

(b) <u>Horizontal and Vertical Alignment</u>

Reducing radius (or increasing curvature) or horizontal curves, especially below values of 450 to 600 metres results in higher crash rates.

Combinations of horizontal and vertical curves also contribute to increased crash rates, especially where combined conditions exceed moderate values of 450 metre radius and 4% gradient.

The cost-effectiveness of such measures was found to be positive.

Evidence to support the relationship that increasing gradient alone contributes to increased crash rates was found to be weaker or perhaps inconclusive.

Much of the analysis was again based on American research. This needs to be similarly researched in Australia to be certain of crash costs and benefits from crash savings which would realistically accrue to road improvement schemes.

(c) Road safety is improved where adequate <u>Superelevation and Crossfall</u> are maintained to provide proper drainage.

Only one study provided evidence of crash reductions attributable to increasing crossfall. The cost-effectiveness is considered to be low in terms of the comparison with minor crossfall changes. However, the resulting road characteristics will improve for other reasons thus producing a positive benefit. Large changes in crossfall do not appear to be cost-effective.

It is considered that it is impracticable to evaluate independently superelevation and horizontal alignment.

(d) Areas of further research have been identified above. Where Australian research is undertaken, study specifications will need careful consideration to ensure that robust data is obtained which can be applied directly to the tasks of assessing attributable crash reductions and cost of improvements. It is important that controlled data sets are obtained (as per Kihlberg and Tharp (1968)) where individual effects can be independently assessed.

10.0 ROADSIDE HAZARD MANAGEMENT

The analysis of roadside hazards has three main aspects:

(i) What constitutes a roadside hazard?
(ii) What can be done about it?
(iii) What are the most effective measures?

Considerable research has been undertaken on the first two aspects both in Australia and elsewhere. On the other hand, the difficulties in defining many of the factors involved in effectiveness mean that few conclusions have been drawn about the third aspect. This chapter attempts to bring to a common base those conclusions which have been drawn and to indicate where further research would be valuable.

10.1 ROADSIDE HAZARDS

10.1.1 Lateral Location

An object or environment may be regarded as a roadside hazard when it is within a certain distance of the edge of the travelled carriageway. From research in Canada, Cooper (1980) has produced lateral encroachment distribution curves (Figure 10.1). The graph shows that two thirds of encroaching vehicles will not exceed 9 m from the carriageway edge. The raw data has been adjusted to allow for unreported encroachments (thus adjusted encroachments of 1 ft (0.3 m) represent 130% of reported 1 ft encroachments).

A slightly different approach was taken by Heulke and Gikas (1967) who plotted the distance to obstacles actually hit in crashes (Figure 10.2). This naturally gives smaller distances than the Cooper research and gives between 82% and 97% of off-road crashes occurring less than 9 m from the carriageway. In particular the proportion of fatalities within 9 m was 90%. Practice in the U.S.A. (AASHTO, 1977) is thus to define a clear zone to 9 m from the travelled carriageway edge. Within this zone slopes are graded to a maximum steepness of 6:1 and any obstacles are

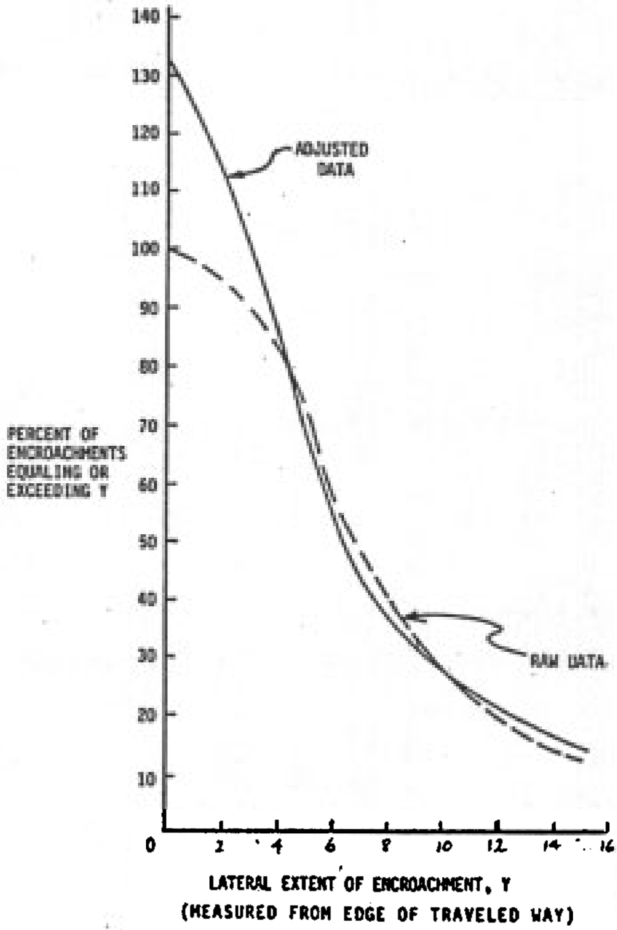


Figure 10.1

Adjusted Lateral Encroachment Distribution

Source: Cooper (1980)

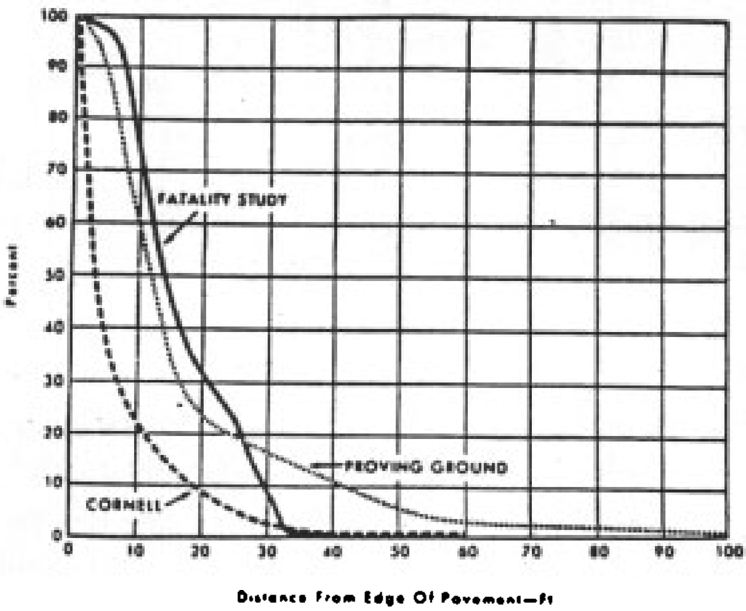


Figure 10.2

Distribution of Impacted Roadside Obstacles vs. Distance from Edge of Roadway (Heulke and Gikas, 1967)

Source: Heulke & Gikas (1967)

removed or, as a last resort, protected with guard rails. Road signs and lighting columns within the zone are provided with breakaway or frangible bases.

Current Australian Guidelines are based on the work by Troutbeck (1983). His concept of minimum and desirable clear zones has been modified by Cunningham (1985) to relate to vehicle flows, with less than 1000 AADT equivalent to the minimum and greater than 4000 AADT equivalent to the maximum (Figure 10.3). Above 96 km/h and 4000 AADT, this gives the same 9 m as the AASHTO practice in the U.S.A.

Since curves of 600 m or less have about 8 times as many crashes as straight sections of road (Boughton 1976) AASHTO recommends clear zones of up to 45 m on the outside of 200 m radius curves. The NAASRA (1986) guidelines propose a doubling of the appropriate clear zones for all curves sharper than 600 m radius (Figure 10.3).

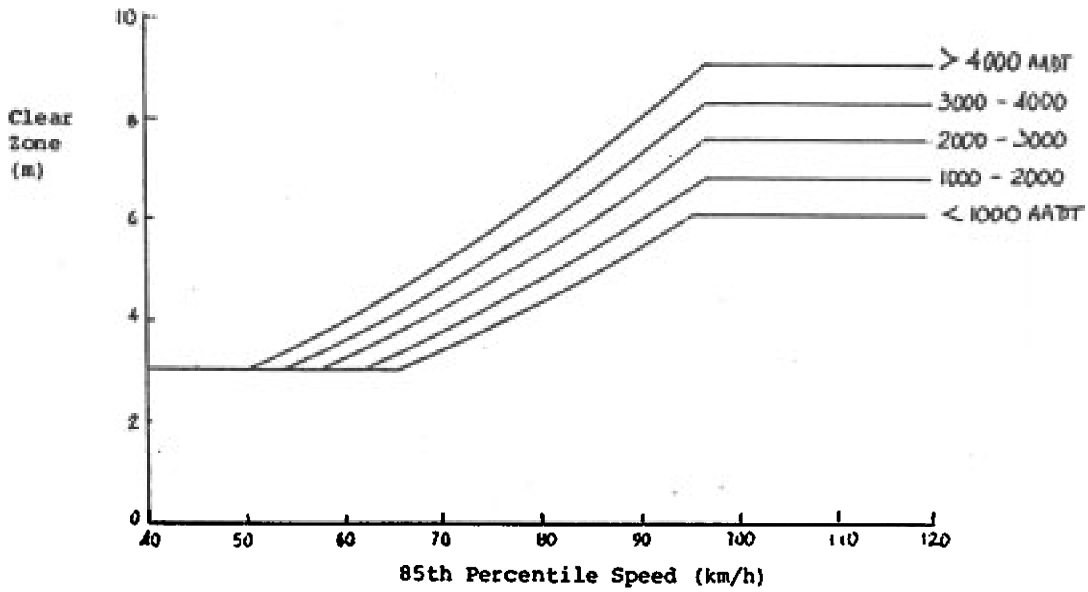


Figure 10.3

Required Clear Zone

Source: Cunningham (1985)

10.1.2 Severity Indices

Roadside hazards represent a variety of levels of danger to encroaching vehicles and their occupants. AASHTO (1977) have categorised the various hazards into the severity indices table shown as Table 10.1, together with the severity of crashes for each index (Table 10.2). However, these crash severity figures represent an average of very widely ranging data. Similarly, the average crash costs refer to the American situation in 1977.

<u>Table 10.1</u>

Severity Indices

Index	Hazard Description
1.	Crash cushions Standard Guideposts (100 x 50)
2.	Mountable kerbs Breakaway sign posts
3.	Small trees Non-mountable kerbs Safety barriers with standard BCTs Low, flat embankments Retaining walls Old guideposts (100 x 100)
4.	Non-breakaway single support signs Post and cable safety barrier High embankments
5.	High steep embankments Safety barrier with substandard end treatment
6.	Safety barrier of sub-standard desi[illegible] and su[illegible]rd end treatment Safety barrier without increased stiffness near a structure
7.	Utility poles Multi-post sign support Luminaire Exposed headwall of culvert Large trees
9.	Bridge piers and end posts Elevated gore areas Exposed end of rock face

Sources: AASHTO (1977)

Table 10.2

Crash Severity Index and Crash Costs (from AASHTO 1977)

Severity Index	Per Cent PDO Crashes	Per Cent Injury Crashes	Per Cent Fatal Crashes	Total Average Crash Costs ($)
0	100	0	0	700
1	85	15	0	2095
2	70	30	0	3490
3	55	45	0	4885
4	40	59	1	8180
5	30	65	5	16710
6	20	68	12	30070
7	10	60	30	66070
8	0	40	60	124000
9	0	21	79	160000
10	0	5	95	190000

Source: AASHTO (1977)

An important feature contributing to the proportion of objects hit is the frequency of occurrence. This factor varies substantially between areas and between urban and rural environments.

Between 1978 and 1982, the type, number and severity of run off road (ROR) casualty crashes in rural Victoria were as shown in Table 10.3. Figures from 1986 (Charlesworth) have been incorporated on an equivalent basis for property damage only (PDO) crashes in order to give crash severity figures. It is widely recognised that PDO crashes are severely under reported. To enable comparisons to be made with estimated crash occurrence calculations, a modified severity index is also shown which assumes that two PDO crashes are not reported for each one that is. These are shown in Table 10.4.

Table 10.3

Summary of Rural Crashes by Type and Object Hit Where Vehicle Ran-Off-The-Road and Struck Fixed Object

	Fatal		Hospitalisation		Medical		Total Casualty	
	No.	%	No.	%	No.	%	No.	%
Trees/Shrubs	229	54	1,122	49	489	40	840	47
Bridge	13	3	86	4	35	3	134	4
Embankments	26	6	268	12	264	22	558	14
Fences/Walls	32	8	212	9	120	10	364	9
Poles	20	5	132	6	59	5	211	5
Safety Rails	8	2	42	2	29	2	79	2
Guide Posts	68	16	279	12	133	11	480	12
Traffic/Signs/ Signals	3	1	30	1	15	1	48	1
Others	21	5	118	5	76	6	216	6
TOTAL	420	100	2,289	100	1,221	100	3,930	100

Source: Sanderson (1984)

Table 10.4

R.O.R. Crash Severity by Type

Severity	Fatal	Severity % Hosp.	Medic.	PDO	Severity Index	Modified Severity Index
Trees/Shrubs	8	41	18	32	28.9	17.6
Guide Posts	8	34	16	42	26.1	14.4
Bridges	5	34	14	47	21.2	11.2
Utility Poles	5	31	14	51	19.3	10.3
Guide Posts (PPK)*	3	36	20	41	18.7	10.6
Safety Rails	5	25	17	53	17.7	9.2
Others	4	25	16	55	16.8	8.3
Embankments	3	28	28	42	15.7	9.1
Fences/Walls	3	19	10	68	12.1	5.9
Traffic Signs/ Signals	2	17	9	72	10.0	4.8
Overall	5	28	16	51	18.6	9.8

* Crash severity where guide post is only hazard hit (Pak-Poy and Kneebone 1986)

The high figures for guide posts are due to the practise of recording the first item to be struck, rather than that which had the most effect. Thus a vehicle which hits a guide post and then, for example, a tree is listed as having hit a guide post. Pak-Poy and Kneebone (1986) reported that guide posts caused less than 2% of injuries and damage in their own right.

The various hazards have been ranked by average cost per reported crash and per estimated crash in Table 10.5.

Table 10.5

Average Cost ($) Per Crash by Type of Hazard

	Reported	Estimated
Trees/Shrubs	75100	45800
Bridges	55100	29100
Utility Poles	50200	26800
Guide Posts*	48600	27600
Crash Barrier	46000	23900
Other	43700	21600
Embankments	40800	23700
Fences/Walls	31500	15300
Traffic/Signs/Signals	26000	12500
Overall	48400	25500

* Crash severity where guide post is only hazard hit (Pak-Poy and Kneebone 1986)

10.1.3 Encroachment Frequencies

Glennon and Wilton (1974) found the following encroachment frequencies for rural roads:

2-lane	E_f	$= 0.00038V_2$
Undivided 4-lane	E_f	$= 0.00023V_2$
Multi-lane divided	E_f	$= 0.00014V_1$

V_1 is a one way AADT, in which case E_f is only for one side of the road. V_2 is a two way AADT, in which case E_f is for both sides of the road.

Cooper (1980) gave adjustment factors for Horizontal and Vertical Alignment (Table 10.6).

Table 10.6

Encroachment Frequency Adjustment Factors for Horizontal and Vertical Alignment

	Encroachment Location with Respect to Curve: Inside		Outside	
Curve Radius (m)	Uphill or Moderate Downhill Grade (LT 2%)	Steeper Downhill Grade (GT 2%)	Uphill or Moderate Downhill Grade (LT 2%)	Steeper Downhill Grade (GT 2%)
600	1.00	0.80	1.00	0.80
300-600	1.24	2.06	2.76	4.60
300	1.98	4.00	4.42	9.00

Source: Cooper (1980)

Willett (1981) compared alignment to crash occurrence in W.A. as shown in Table 10.7.

Table 10.7

Crash to Alignment Proportions

	Crest/Slope % Lengths	Crest/Slope % Crashes	Level % Lengths	Level % Crashes	Total % Lengths	Total % Crashes
Curve	2	15	10	25	12	40
Straight	10	15	78	45	88	60
TOTAL	12	30	88	70	100	100

Source: Willett (1981)

The variation factor from the mean is thus as follows (Table 10.8).

Table 10.8

Variation Factor From Mean

	Crest/Slope	Level	Total
Curve	7.5	2.5	3.3
Straight	1.5	0.6	0.7
TOTAL	2.5	0.8	1.0

Thus a location on a curve on a slope is 7.5 times as likely to incur an crash than the mean.

The work of Cooper in Canada (Section 10.1.1) distinguishes between the inside and outside of curves. These proportions have therefore been used in conjunction with Willett's Australian results to give the following encroachment adjustment factors (Table 10.9).

Table 10.9

Encroachment Adjustment Factors
For Curves LT 300 m Radius and/or Slopes GT 2%

	Crest/Slope	Level	Total
Outside Curve	10.4	3.5	4.6
Inside Curve	4.7	1.6	2.0
Straight	1.5	0.6	0.7
TOTAL	2.5	0.8	1.0

10.2 ROADSIDE HAZARD MANAGEMENT

The occurrence of crashes involving roadside hazards can be dealt with in the following way:

(i) removal or relocation to outside a "clear zone";

(ii) reduction of effects of collision;

(iii) provision of guard barriers.

10.2.1 Hazard Removal

Certain categories of hazards can be removed or relocated outside the clear zone. These include:

Vegetation (trees, shrubs);
Poles (electric, Telecom);
Street Furniture (mail boxes, telephone booths, substations).

Other categories by their nature must be within the clear zone. These include:

Highway lighting posts;
Traffic Engineering (signal supports, sign supports);
Road Features (kerbing, gore structures, level crossing furniture);
Bridges (abutments, supports, parapets, rails).

There are some items such as embankments and cuttings which could be placed outside the clear zone, but at high cost. Similarly, many items within the clear zone could be positioned further from the carriageway.

10.2.2 Reduction of Collision Effects

During the 1970's new types of lighting and traffic poles were developed with either slip-type fittings or with a weak zone.

These are generally referred to as breakaway poles. The word "frangible" is sometimes used as a general term and sometimes only for poles with a weak zone (usually achieved by drilled holes or sawcuts). In this report frangible will be used in the latter sense. In America, slip base poles are now standard for rural roadside lighting poles and traffic signs. The crash severity effect has been so dramatic that there is little statistical data to utilise. More common are statements such as "vehicle collisions with luminaire supports have been reduced to "fender-bender" status. Fatalities in such occurrences have become almost non-existent". (Tamanini, 1981).

There have been problems with two types of rigid poles. There is a risk of electrocution if electricity poles are brought down by cars and no satisfactory method had been found for wooden breakaway posts. Both of these problems were addressed in 1986 by Ivey and Morgan. They described a system of providing a metal slipbase for wooden poles together with an upper hinge point and overhead guys.

The result of a collision is shown in Figure 10.4. The injury severity reductions were startling. At an impact speed of 60 mph (100 km/h) the likelihood of serious injury from a rigid wooden pole was 76.5%, while for the breakaway pole it was 0.5%. The equivalent figures for 40 mph (60 km/h) and 20 mph (30 km/h) were 22.4% to 0.6% and 2.5% to 0.4%. It is thus now practical to fit breakaway posts to wooden utility poles and to steel electricity supply posts.

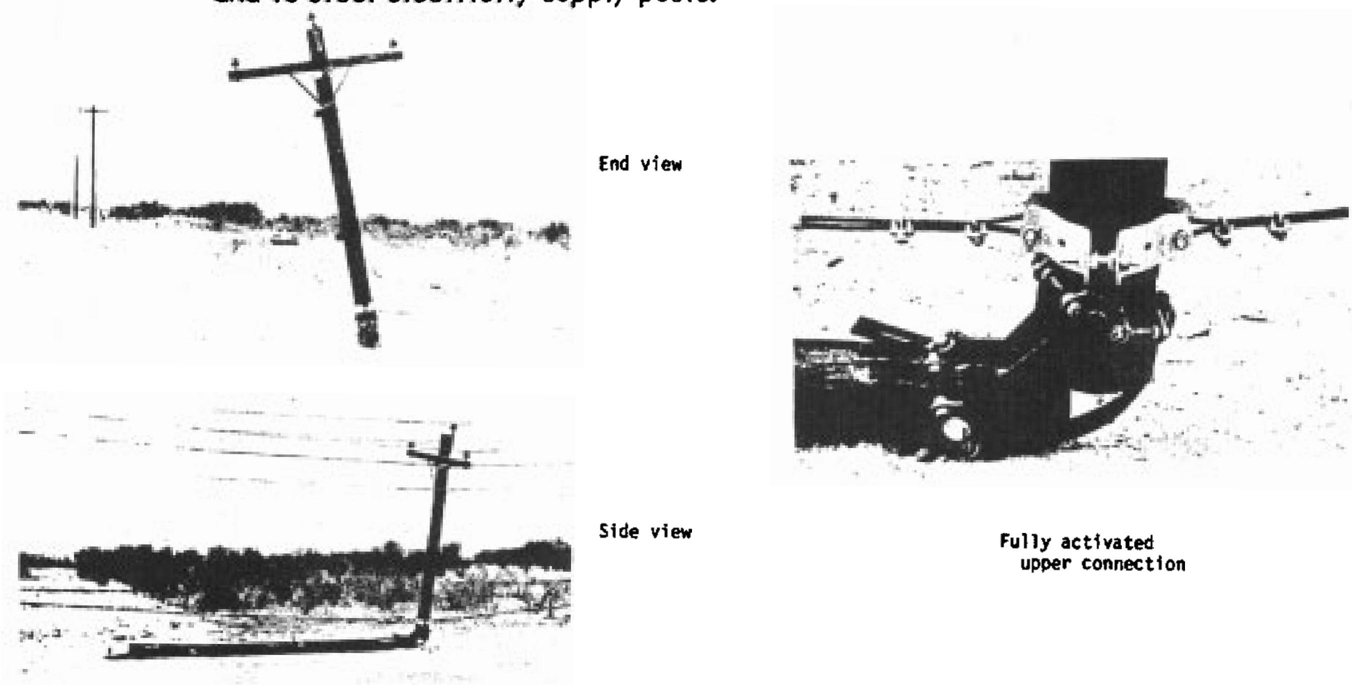


Figure 10.4

HBS-Modified Utility Pole After a High-Speed Collision (Test 4859-3)

Source: Ivey & Morgan (1986)

10.2.3 Provision of Guard Rails

Since guard rails represent a hazard in themselves, they should only be used where neither of the above options are practical and where the hazard being guarded is likely to have greater crash costs than the barrier. In America where guard rails have been widely used, they were associated with 11% of roadside hazard fatalities in 1978 (NHTSA, 1979). The equivalent rural Australian figure for 1978-82 was 2%.

10.3 COST-EFFECTIVENESS

The cost-effectiveness of roadside hazard management is dependent on the number of crashes which can be prevented, the number of crashes which can be reduced in severity, the cost of crashes and the cost of the safety measure. It is therefore necessary to know the number and severity of crashes caused by each hazard. Sanderson and Fildes (1984) report that, in rural Victoria, "ran off road" (ROR) casualty crashes represented 35% of all casualty crashes. Of the ROR crashes, in 62% of cases the vehicle struck a fixed object. Thus of all rural casualty crashes a roadside hazard is the first object struck in 22% of cases. This figure was also found for South Australia by Boughton and Milne (1977). There are also a number of crashes which occur on the road where a vehicle subsequently leaves the road and strikes a roadside hazard. There is thus a substantial potential crash cost reduction from roadside hazard management.

10.3.1 Occurrence of Crashes

The likelihood of a vehicle which leaves the road hitting a particular hazard has been assessed by Troutbeck (1983) building on the work detailed in Section 10.1.

He produced the following equation for the collision frequency (C_f) with a particular hazard:

$$C_f = \frac{E_f}{1000}\left[L.P(A)+4.72P(A-0.44)+5.14\sum_{j=1}^{W}P(A+j-0.5)+4.72P(A+W+0.44)\right]$$

Where: E_f is the encroachment frequency/km/yr

L is the length of the hazard (m)

A is the distance from the hazard to the travelled way (m)

P(A) is the probability of errant vehicles travelling further than X m from the travelled way

W is the width of the hazard (m).

The value of the mean impact angle is taken as 11 degrees and the width of the errant vehicle as 1.8 m.

The curve used to find the probability of an errant vehicle travelling further than A m from the travelling way is that shown in Figure 10.5.

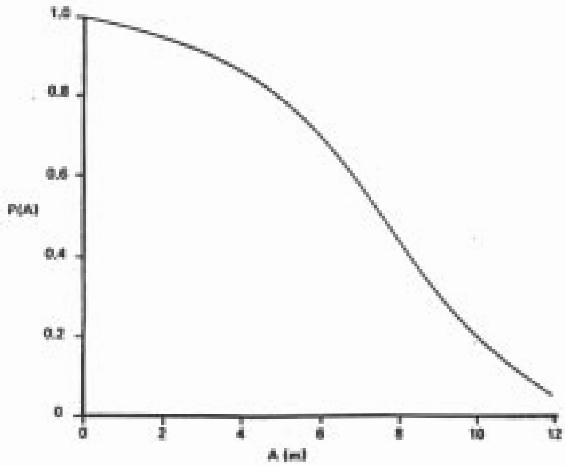


Figure 10.5

Probability that an Errant Vehicle Travel Further Than A (m) from the Travelled Way

Source: Troutbeck (1983)

The equation can be expressed as:

$$C_f = E_f \left(X(L,A) + B(A,W) \right)$$

Where: X (L,A) is a function of L and A

and B (A,W) is a function of A and W

These functions are shown graphically in Figure 10.6.

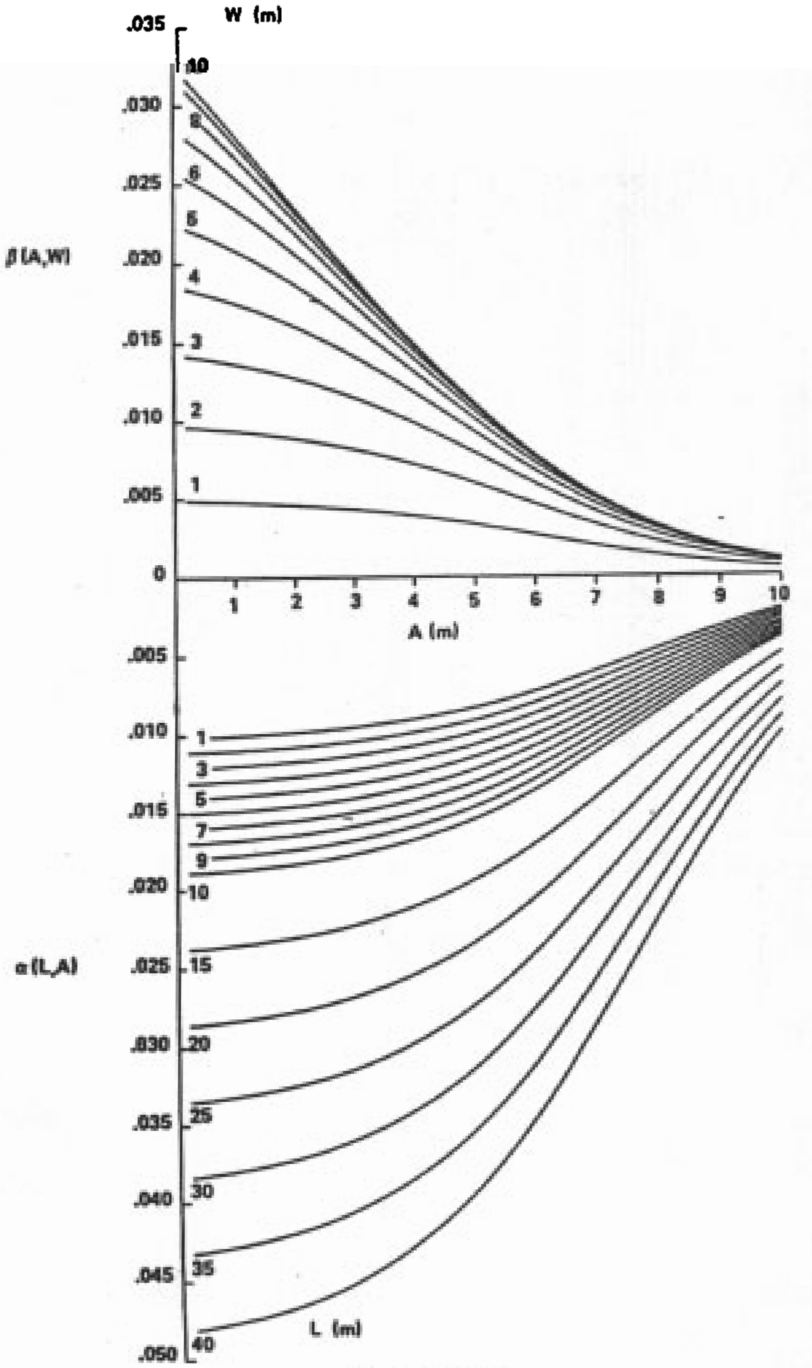


Figure 10.6
Coefficients α(A,L) and β(W,A)

Source: Troutbeck (1983)

If W is small (e.g. utility pole) or L is greater than about 40 m (e.g. continuous guard rail) the equation can be approximated to:

$$C_f = \frac{E_f (L + 10) P(A)}{1000}$$

Where a group of hazards occur together (e.g. a clump of trees) they should be considered as one hazard.

The encroachment frequency should be adjusted according to the alignment factors shown in Table 10.4 and the following features:

Characteristic		**Factor**
Shoulder width:	3.5 m or greater	1.00
	2.5 m	1.10
	1.5 m or less	1.20
Weather:	Normal	1.00
	Ice or fog prevalent	1.40

Example: A 500 mm dia tree 4.5 m from the travelled carriageway on the outside of a 200 m radius bend on a 2-lane road with 2.5 m shoulders and an AADT of 2000 vehs.

$E_f = 0.00038 \times 2000 = 0.76$

$(L, A) = 0.008$

$(A, W) = 0.002.$

$C_f = 0.0076$

geometry factors = 3.50 (curve), 1.10 (shoulders)

$C_f = 0.029$ collisions/year

10.3.2 Cost of Crashes by Hazard Type

The costs of crashes for a particular hazard on a yearly basis may be obtained by two means.

The first is to determine the total number and severity of crashes involving a particular type of hazard and the number of those hazards which occur on the whole of the network covered by the crash reports. The number and severity of crashes is generally fairly well known but the number of any particular hazard is rarely known. For some hazards (e.g. bridges) the information could probably be obtained fairly readily but for others (e.g. trees) it would be extremely difficult to obtain.

Table 10.10 shows a ranking of hazards based on the crash severity index and the percentage of crashes involving each type of hazard (the occurrence) (Sanderson and Fildes (1984)). This combined index shows that in rural areas, trees are nearly four times as much of a problem as the next hazard. Even this next hazard (guide posts) is overreported, as detailed in Section 10.1.2. Eliminating the overreporting makes embankments the next major hazard at one fifth the "dangerousness" of trees.

Table 10.10

R.O.R. Crash Severity and Occurrence by Type

	Severity Index	Occur-ence	Occur. x Severity	Modified Occur.	Modified Occur. x Severity
Trees/Shrubs	28.9	38%	11.0	42%	12.1
Guide Posts	26.1	12%	3.0	-	-
Guide Posts (PPK)*	18.7	-	-	2%	0.5
Embankments	15.7	13%	2.1	14%	2.2
Fences/Walls	12.1	16%	1.9	18%	2.2
Utility Posts	19.3	6%	1.2	7%	1.4
Others	16.8	7%	1.1	8%	1.3
Bridges	21.2	4%	0.7	5%	1.1
Safety Rails	17.7	2%	0.4	2%	0.4
Traffic Signs/ Signals	10.0	2%	0.2	2%	0.2
Overall	18.6	100%			

* Crash severity where guide post is only hazard hit (Pak-Poy and Kneebone 1986)

Source: Sanderson & Fildes (1984)

The second approach is the numerical approach detailed in Section 10.3.1, combined with the severity index in Section 10.1.2.

10.4 COST-EFFECTIVENESS CONCLUSIONS

As mentioned in Section 10.3, there is a wide range of Benefit Cost Ratios for each hazard due to its location.

Using the example in Section 10.3.1, there is a collision frequency of 0.029 collisions/year with the tree in question. From Section 10.1.2, a large tree has a severity index of 28.9 and thus an average cost per estimated crash of $45,800. The theoretical crash cost of that tree is thus $1300/year.

The cost of removing such a tree is estimated as being in the range $200-$500. Taking a figure of $400 for this example, a benefit period of 10 years and a discount rate of 10% for the benefit, the Net Present Cost (NPC) is $400, and Net Present Benefit (NPB) is $8600.

The benefit cost ratio (BCR) is thus 21:1.

It has been demonstrated that no one fixed benefit can be allotted to a particular type of roadside hazard. The range of variables (distance from carriageway, highway alignment, shoulder width, climatic variables) is too wide and has too many unknowns to be able to produce even a mean value. The example given, however, shows that substantial cost benefit ratios are available and the number of crashes involving roadside hazards is large enough to warrant continued action.

With a 1987 roadside hazard crash having an average cost in the range $25000 to $48000, the BCR may be large even with low flows. For the example given the BCR would have been greater than 1 for a flow as low as 120 veh/day.

Where utility poles have to be within about 9 m of the carriageway, there is substantial justification for making them breakaway poles, even for electricity poles. The BCR would be about two thirds of that for removal of large trees.

Removal of a tree 8 m from a straight level road with 1000 AADT and 3.5 m shoulders would have a BCR of 0.9.

In certain circumstances it may be cost-effective to remove crash barriers where alternative improvements (e.g. slip bases) are available.

10.5 FURTHER RESEARCH

The following research would thus appear to be appropriate:

(a) Australian research to verify the rural run off road encroachment formulae;

(b) research into the high severity of guide post crashes;

(c) investigation of more widespread use of slip bases on poles in rural areas;

(d) more comprehensive research on crash data to distinguish relative severities of hazard crashes and guard barrier crashes;

(e) research on enbankment crash severities;

(f) investigation of removal of guard barrier and implementation of more appropriate action (e.g. removal of hazards or slip bases on poles);

(g) to attempt to utilise actual crash data more often, rather than using the theoretical formulae, would require a large data base of existing roadside hazards related to alignment and lateral location to allow exposure ratios to be calculated for lengths of roads. This would be a major undertaking on a national basis but could be started for major rural roads.

11.0 MEDIANS AND BARRIERS

Medians are provided to give better separation of vehicles travelling in opposite directions. Head-on collisions have one of the highest severity indices of all types of crash. The separation may be achieved in three ways:

(i) a large distance between the carriageways;
(ii) a physical guard barrier;
(iii) a combination of the above.

Medians are only appropriate on 4 (or more) lane highways, apart from very short lengths, usually at junctions.

The purpose of a guard barrier is to protect drivers from having collisions with hazards which would cause more severe crashes than the barrier itself. Since guard barriers themselves have a high crash severity, the average cost of a collision with a barrier is estimated at $23,900 (Section 10.2). As the likelihood of a collision with a barrier is greater than with the object it is protecting (due to its inherent greater length and proximity to the carriageway), it follows that the hazard has to have a substantially higher crash cost before a guard rail is justified.

11.1 WIDE MEDIANS

The likelihood of a vehicle encroaching on the opposite carriageway can be estimated from the work done by Cooper (1980) or by Heulke and Gikas (1967) as detailed in the section concerning roadside hazards. Thus with a median width of 9 m, between 70% and 90% of vehicles encroaching on the median would not reach the other carriageway. It has been suggested (NAASRA, 1986) that a clear width of about 7 to 10 m "for each direction in the case of medians" should be provided for high volume roads. This allows for vehicles travelling in opposite directions to simultaneously lose control and enter the median at the same place, a very remote possibility.

Research in America (Kihlberg & Tharp, 1968) showed that 4-lane highways with no access control had 23% fewer crashes per km length with medians compared to similar roads without medians. On a crashes per million vehicle kms basis, the equivalent reduction was 20%. It should be noted, however, that the crash severity index (see Chapter 7) increased by 7%. The overall savings in crashes and crash costs were as follows (Table 11.1).

Table 11.1

Crash Cost Savings

	No Median			With Median			Savings			
	No.	Severity	Cost $	No.	Severity	Cost $	No.	%	Cost $	%
Crashes/ Year/km	13.3	14.7	508326	10.3	15.7	420446	3.0	23	87880	17
Crashes/ Million Vehicle km	2.48	14.7	94786	11.99	15.7	81232	0.49	20	13554	14

These figures are on roads with a mean AADT of about 14000 vehicles. Clearly, it would not be cost effective on crash savings alone, to convert an existing 4-lane undivided highway to a 4-lane divided highway. However, it is worth considering the difference for new construction. It is estimated that the additional cost of providing a 9 m median on a new highway is of the order of $80,000 per km. For the mean flow in Table 11.1 (i.e. 14000 veh. AADT), the BCR is 7.4 at a benefit discount rate of 10% p.a. For an AADT of 8000 vehs., the BCR would be 3.3.

11.2 GUARD BARRIERS

Guard barriers are entirely dependant on a reduction in crash severity for their benefits since they are of necessity both closer to the road and longer than the hazard they guard. They thus have a higher crash rate than the hazard in question. Since existing guard barriers have a high crash severity in their own right (17.7, see Table 10.4), they are only cost effective in guarding hazards with a higher crash severity index.

The research into guard (or safety) barriers up to the early 1980's has been assessed by Troutbeck (1983). He points out that, since it is the difference in crash severities rather than absolute severities that is relevant, the way in which this is expressed becomes important. Various researchers (AASHTO (1977), Glennon & Tamburri (1967), Fox et al (1979), UK DTP (1984) and Federal Office of Road Safety (FORS) (1984)) have used different crash costs for different severities of crashes which have given different severity factors (related to PDO as 1) as shown in Table 11.2.

Table 11.2

Crash Severity Factors Based on Cost

	Glennon (67)	AASHTO (77)	Fox (79)	UK DTP (83)	Modified FORS (87)
Fatal	25	286	64	2648	157
Serious	) 6	) 14	) 4	123	35
Slight	)	)	)	3	4
P.D.O.	1	1	1	1	1

Note: The UK DTP figures gave costs only for slight injury, serious injury and fatal crashes. To allow comparisons to be made, a nominal value of 3 has been allocated to the slight injury costs.

The other factor in deciding the benefits of installing guard barriers is the severity index of crashes. This has been defined in Chapter 10 as the sum of the products of the proportion of each severity of crash and its severity factor. Thus if it is found that collisions with trees give 8% fatal crashes, 41% serious injury crashes, 18% minor injury crashes and 32% property damage only, then the severity index (SI) is calculated as below (using FORS crash severity factors from Table 11.2).

$$SI = 0.08 \times 157 + 0.41 \times 35 + 0.18 \times 4 + 0.32 \times 1 = 28.9.$$

By multiplying the Severity Index by \$2600, an average cost of that type of crash is found (in the example \$75140).

Not only do the severity factors vary very widely in the literature but so do the percentages for the crash severities. For example, AASHTO (1977) and Victorian (Charlesworth, 1987) figures for trees greater than 150 mm dia are given below:

Table 11.3

Severity of Crashes with Trees

	AASHTO	Victoria
Fatal	30%	8%
Injury	60%	60%
PDO	10%	32%

Source: AASHTO (1977) and Charlesworth (1987)

Hazards within the clear zone defined in Chapter 10 need to be considered for provision of guard rail. However, other methods such as removal of the hazard should be considered first. The benefits of removal will always be higher than those offered by provision of a barrier.

11.2.1 Barriers for Non-Traversable Hazards

Certain hazards are considered (AASHTO, 1977) as "non-traversable hazards" and the cost of their removal likely to preclude that action. These include:

rough rock cuts;
large boulders;
streams or permanent bodies of water deeper than 0.6 m;
opposing traffic (AADT greater than 2500 veh/day, one way);
abrupt drops in roadside (greater than 1.6 m).

11.2.2 Barriers for Fixed Hazards

The next category of hazards are the fixed hazards such as those listed in Table 10.4. For these it is often practical to remove the hazard or reduce its severity index (e.g. fitting utility poles with slip bases). A BCR can be worked out as detailed in Chapter 10 and compared with the BCR of installing guard rail. Guard rail will give a benefit due to reducing crash severity but this will be offset by an increase in crash rate due to both increased length of exposure and greater proximity to the roadway than the hazard. These effects can also be quantified as shown in Chapter 10. Australian figures seem to show that a guard rail is rarely the most cost effective measure for fixed hazards on a crash basis.

11.2.3 Barriers for Embankments

The third category of hazards are embankments. An overall severity index (15.7) for Victoria is given in Table 10.4. This can be compared to the data of Glennon and Tamburri (1967) in Table 11.4.

Table 11.4

Comparison of Glennon & Tamburri Data with Victorian Data

Range of Embankment Heights (m)	Range of Embankment Slopes	Crashes Fat.		Inj.		PDO		S.I.
0-9.1	3:1	2		27		41		14.3
0-6.1	2:1	13		290		187		18.8
6.1-45.7	2:1	23		264		108		25.5
45.7-152.0	2:1	10		30		4		52.1
G. & T. total		48	(5%)	611	(61%)	340	(39%)	22.6
Victoria totals			(4%)		(56%)		(42%)	15.7
G. & T. Guard Fence		14	(4%)	147	(44%)	170	(51%)	17.8
Victoria Guard Fence			(3%)		(42%)		(53%)	17.7

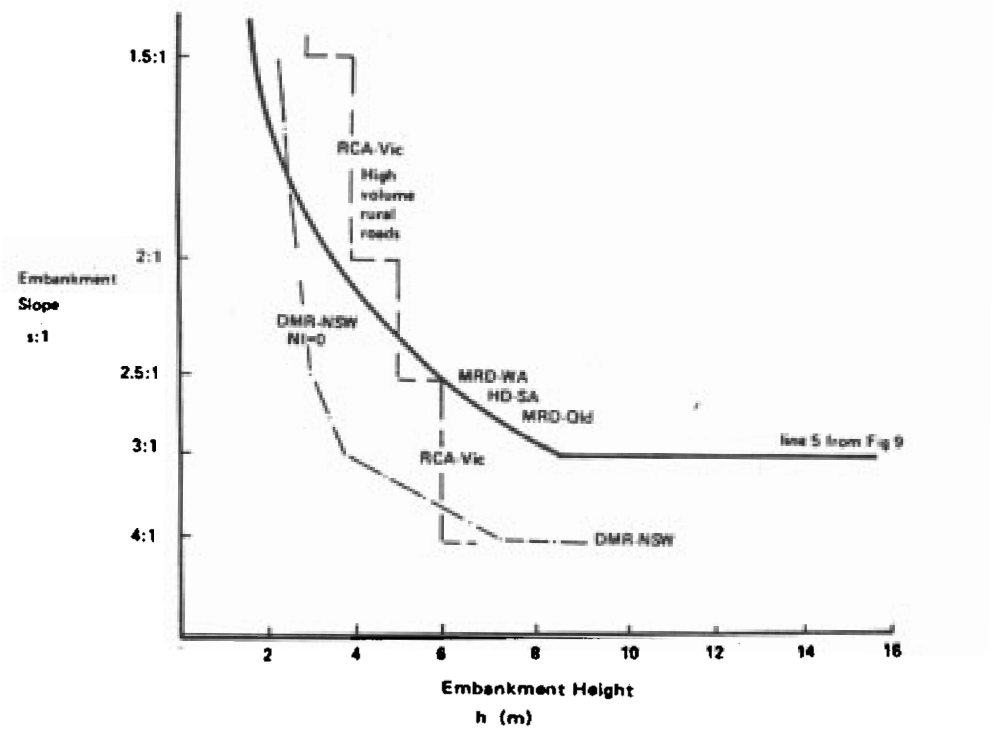
The severity indices for guard fencing as determined by the two studies are almost identical (17.8 v. 17.7). Glennon and Tamburri's overall severity index for embankment (22.6) is considerably higher than that for Victoria (15.7), possibly because of the very high embankments (up to 152 m high) included in the Glennon and Tamburri work. If this highest category is excluded, the overall S.I. is still 21.2 compared to 15.7 for Victoria. In order to give an indication of the type of embankment where guard rail may be an advantage in the Australian context, the S.I.'s for the Glennon and Tamburri data have been reduced to give an overall S.I. equal to the Victorian S.I. These values are shown in Table 11.5 below.

Table 11.5

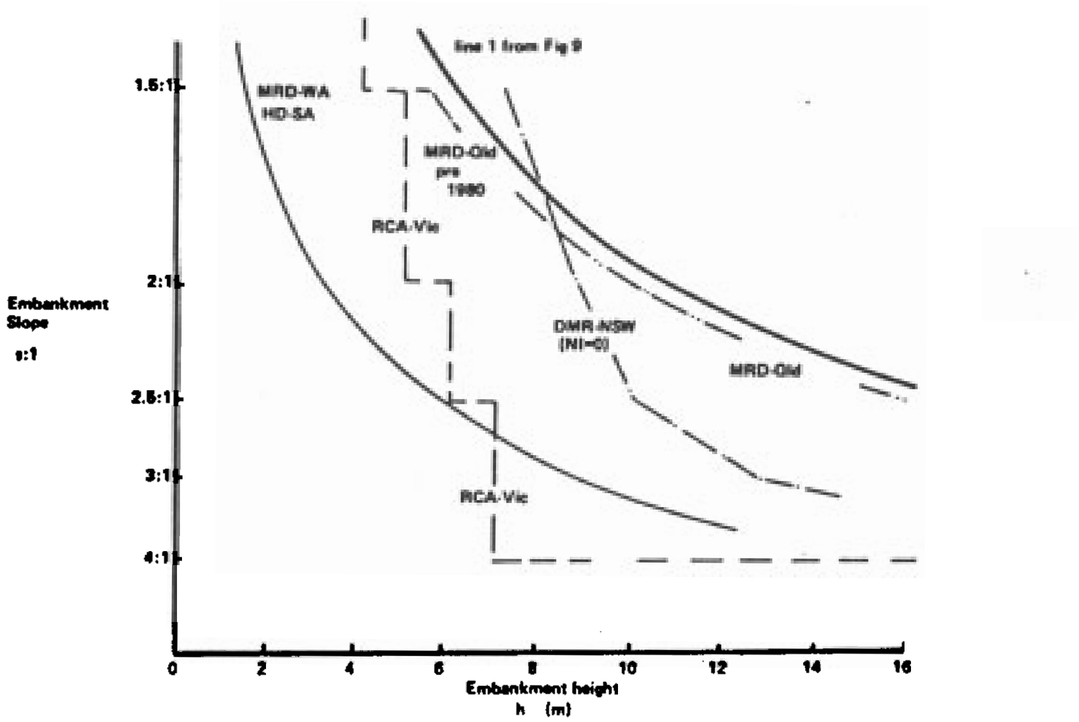
Modified Embankment Severity Rates

Range of Embankment Heights (m)	Range of Embankment Slopes	S.I.
0-9.1	LT 3:1	10.6
0-6.2	GT 2:1	13.9
6.1-45.7	GT 2:1	18.9
Overall		15.7

These values indicate that current Australian warrants (see Figure 11.1) recommend guard barrier at embankments where they could in practice give disbenefits. The existing warrants on the whol merican research and it is in the variation of crash costs and severities that uncertainty is introduced. The methodology used above is only intended to give an indication of the effects of applying Australian data. A further analysis of detailed Australian crash figures for various types of embankment is needed.



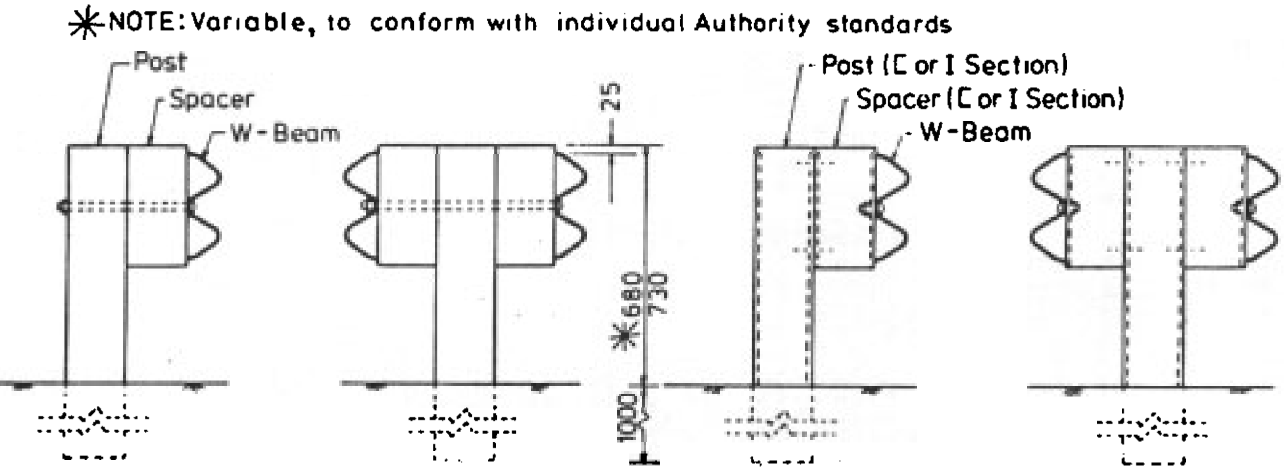
Guidelines for Installing Guard Fences on High Volume Rural Roads



Guidelines for Installing Guard Fences on Low Volume Rural Roads

Figure 11.1

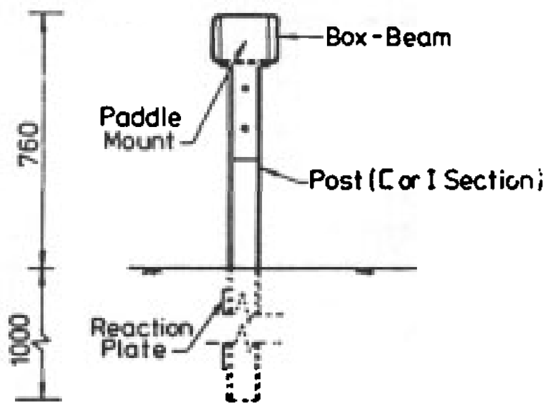
Source: Troutbeck (1983)



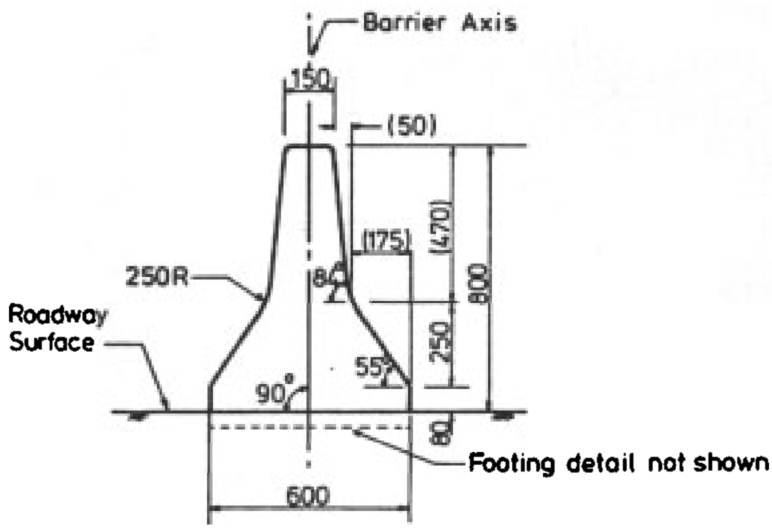
a) Timber Spacers and Posts

b) Steel Spacers and Posts

Blocked-Out Steel W-Beam Guardfence



New York Steel Box-Beam Guardfence



New Jersey Concrete Barrier

Figure 11.2: Barrier Types

Source: NAASRA (1986)

11.2.4 Barrier Types

There are basically three types of barrier currently recognised as appropriate by NAASRA (1986). These are shown in Figure 11.2. Some research has been done in America (Schultz et al, 1986) into the cost-effectiveness of different types of barrier. However, the mix of vehicle types is very important in respect to barrier design and Australian research is needed.

11.3 MEDIAN WITH GUARD BARRIER

The British Transportation and Road Research Laboratory (TRRL) (1974) studied the effect of installing safety fence in the median of the MI motorway. The number of expected and actual crashes for vehicles entering the median were collated for 4 years and are given in Table 11.6.

Using the FORS crash costs and severity factors, the annual cost savings were as follows:

	Expected Crashes			Actual Crashes	
	No.	S.I.		No.	S.I
	732	17.5		836	14.9
Total cost	= 732 x 17.5 x 2600 = $33358000 (4 yrs, 29 kms)		Total cost	= 836 x 14.9 x 2600 = $32307600 (4 yrs, 29 kms)	
Annual cost/km	= $287569			= $278514	

Annual savings/km	= $9055
10 yr NPB	= $58977
Cost	= $70000
BCR	= 0.84

This finding on the relatively narrow (less than 4 m) medians on the busy UK motorway is surprising and warrants further investigation in an Australian environment. Current Australian guidelines recommend a median barrier for all medians of 6 m or less on roads with AADT of 20000 or more and for medians of up to 9 m on roads with AADT of 25000.

Table 11.6

Numbers of Expected[1] and Reported Accidents on 29 km of MI (3-Lane) with Central Safety Fence

Type of Accident	Severity	Number of Accidents Expected[1]	Number of Accidents Reported	% Change
Reserve entered but not crossed	Fatal	8.7	8	
	Serious injury	34.9	44	
	Slight injury	42.6	50	
	No injury	51.0	132	
	Total	137.2	234	70% increase
Reserve crossed but no collision in opposite carriageway	Fatal	2.5	1	
	Serious injury	20.5	4	
	Slight injury	20.2	2	
	No injury	27.0	5	
	Total	70.2	12	83% reduction
Reserve crossed followed by collision in opposite carriageway	Fatal	12.6	0	
	Serious injury	17.0	0	
	Slight injury	6.3	0	
	No injury	32.0	1	
	Total	67.9	1	98% reduction
Reserve not entered	Fatal	14.7	26	
	Serious injury	91.8	115	
	Slight injury	100.3	144	
	No injury	250.2	304	
	Total	457.0	589	29% increase
All Accidents	Fatal	38.5	35	9% reduction
	Serious injury	164.2	163	1% reduction
	Slight injury	169.4	196	16% increase
	No injury	360.2	443	33% increase
	Total	732.3	836	14% increase

(1) Number estimated if fence had not been erected
Source: TRRL (1974)

Considering only median crossover crashes gives a BCR of 6.4 but when guard barriers are installed on relatively narrow medians, there is an increase in crashes within the carriageways which reduces the BCR to less than 1 as above.

11.4 WIDE MEDIANS WITH GUARD BARRIER

To install guard barriers on wide, level and clear medians will usually produce an actual disbenefit. For example, from Figure 10.1, 73% of encroaching vehicles could be expected to stop within a 10 m median. However, if a central barrier is installed, only 30% of encroaching vehicles could stop before hitting the barrier. The severity index for cross-over crashes would thus have to be greater than 2.6 times that for barrier collisions. Research in England (TRRL, 1974) showed severity indices of 27.9 and 13.4 respectively, i.e. only 2.1 times. The benefit cost ratio would thus not only be less than 1 but actually be negative.

The severity index for non level median (e.g. where the carriageways are on separate embankments) may be obtained from Section 11.3. For a median with hazards, the benefit of hazard removal can be estimated from Chapter 10 and the benefit (or disbenefit) of providing guard barrier can be obtained from Section 11.3).

11.5 CRASH ATTENUATORS

Crash attenuators (also known as crash cushions) can be used in situations where head-on collisions with hazards would otherwise be difficult to avoid. These include bridge piers on narrow medians and gore areas with hazards (especially gaps between parallel bridges). There are a variety of types recognised by NAASRA (1986) which have been shown in the U.S. to give good performance (Pigman et al, 1985). Pigman considered 127 reported crashes and described crash severities. Using the FORS (1984) costs, these gave an average crash severity of 11.0, well below that of most hazards. They would not be expected to reduce the number of crashes but could substantially reduce the severity. Individual hazards would need to be assessed to give cost-effectiveness figures.

11.6 FURTHER RESEARCH

Suggested further research into guard barriers has been given in the further research section (10.5) of roadside hazards. It is also recommended that research be undertaken into the effectiveness of guard barrier designs for vehicles on Australian roads, now and predicted for the future.

Crash attenuators have shown worthwhile benefits in the U.S. It is likely that their usefulness in rural Australia is limited but any installations which are made should be carefully analysed from the crash viewpoint. In particular, the crash severities before and after need to be researched.

12.0 INTERSECTION TREATMENTS

12.1 GENERAL

There is considerable literature on intersections in urban areas, their crash problems and the remedies for these problems, but urban solutions are not directly transferable to rural areas because of the substantially different environment and traffic behaviour.

A study of rural crashes in South Australia by the National Health and Medical Research Council et al (1985) found that rural crashes were more severe than crashes in Metropolitan Adelaide. It also indicated that while 47.6% of rural "in-town" crashes involved multiple vehicles, 71.9% of rural "out-of-town" crashes involved single vehicles. Another analysis showed that 45.1% of "in-town" crashes occurred at intersections while only 11.1% of rural "out-of-town" crashes occurred at intersections. This seems to indicate that either "out-of-town" intersection crashes are not as significant a problem, which is a little surprising, or these proportions may only indicate the extent of vehicle usage of these locations.

The effect of traffic volumes is complex. Crash rates are more sensitive to changes in crossroad (minor) volume than to changes in major road volume. Low cross road volume intersections have higher crash rates per crossroad vehicle than do high cross road volume intersections. Crash involvement per vehicle is higher for low volumes than for high volumes.

12.1.1 Sight Distance

Across the corner sight distance effects probably apply equally to rural intersections as they do for urban intersections except that the vehicular speeds are higher in the rural area.

The effect of poor sight distance at rural "in-town" intersections in Virginia, U.S.A. was investigated by Hanna et al (1976) and showed that intersections with poor sight distance had 1.33 crashes per million entering vehicles (MEV) compared to the average 232 intersections of 1.13 crashes per million entering vehicles. This would indicate that sight distance improvement could be very cost-effective. Taking a major road flow of 5000 AADT, a minor flow of 2000 AADT and a sight distance clearance cost of $5000 gives a BCR of 5:1.

12.1.2 Sign Control

Hanna et al (1976) indicated that intersections at rural "in-town" locations controlled by Stop and Yield signs (1.08/MEV) had lower crash rates than for traffic signals (1.26/MEV), but there were no comparisons with uncontrolled locations.

Glennon (1979) estimated the crash reduction effectiveness of installing two-way stop signs by using a poisson probability of conflict analysis to estimate the annual number of right angled crashes for various combinations of intersecting traffic volumes on low volume roads (see Table 12.1).

Table 12.1

Expected Annual Crash Reduction of Two-Way Stop Control at Low Volume Road Intersections

Road A ADT	Road B ADT 50	100	200	400
50	.0029	.0058	.0117	.0234
100	.0058	.0117	.0234	.0468
200	.0117	.0234	.0468	.0936
400	.0234	.0468	.0936	.1872

Source: Glennon (1979)

Using an average crash cost of $9,500, Glennon's estimated cost of the sign installation and an estimated cost of a vehicle stop of $0.021 it was concluded that two way stop signs could not be justified even for a 100% reduction of right angled crashes on low volume roads.

On the other hand O'Brien (1976) found that the use of STOP or GIVE WAY signs produced a statistically significant increase (level of significance not reported) after the installation of signs and concluded that unless the minor legs are realigned to prevent high speed crossing movements the use of signs can not be expected to reduce crashes.

An additional feature of STOP signs is that if they are used at locations where visibility is good, drivers will tend to disregard them. This lack of respect can then transfer to locations where visibility is poor (and the STOP sign thus justified on visibility grounds) and actually lead to increases in crashes.

12.1.3 Delineation

Roy Jorgenson (1978) reported that edge line delineation of intersections with edge lines achieved a 62% reduction to crashes.

12.1.4 Rumble Strips

Carsten (1983) concluded that the frequency of crashes at rural locations on secondary roads was independent of the presence or absence of rumble strips. However secondary road intersections that had crash rates in excess of 2.5 crashes per million vehicles entering the intersection showed a reduction to the crash rate following the installation of rumble strips.

Then Zaidel et al (1986) indicated that paint stripes across the road to warn motorists of an approaching intersection had only a minor effect on driver behaviour whereas rumble strips lowered speeds by 40% and reduced the deceleration to a more moderate rate so that few exceeded the 1.3g level. However crash rates were not investigated.

12.1.5 Intersections Type

A large proportion of rural cross intersections function as double "T" junctions with very little traffic crossing the main through route. These intersections have markedly different crash characteristics from intersections where there are significant destinations on each side of the intersection. In addition to this characteristic, the form of the intersection or junction has a significant influence on the crash rates. O'Brien (1976) established with a high level of statistical significance (level of significance not reported) that (a) "Y" type junctions had a higher crash rate than "T" junctions, and (b) cross intersections had nine times the crash rate of "T" type junctions. This study also showed that 90% of crashes at cross intersections occurred in daylight hours and 84% of all crashes were right angled collisions which involved straight through vehicles. At "Y" junctions 62% of collisions involved vehicles approaching at greater than 120 degrees (i.e. virtually head-on).

Therefore "Y" junctions should be converted to "T" junctions and cross intersections with substantial crossing traffic should have the minor roads diverted to create a staggered "T" junction. The stagger needs only be sufficient to eliminate the possibility of a fast through path on the minor road.

This study used vehicle interaction as a measure of exposure and treatment should be considered when the number of crashes in 7 years. Line A in Figure 12.1 below represents two independent "T" junctions, and Line B represents the level at which improvements should be considered.

12.1.6 Turning Lanes

Provisions for through traffic to bypass traffic waiting to turn right at intersections and junctions will obviously influence the crash rate where the volumes using the road are high. The study by Pickering et al (1986) involved a large proportion of locations with major road volumes over 6,000 vehicles per day junctions. In this study junctions with "ghost" (painted) islands to guide traffic had 70% lower crashes with right turning traffic and 70% lower overtaking (head-on) crashes than those without. It also indicated that a wider pavement had much the same benefit.

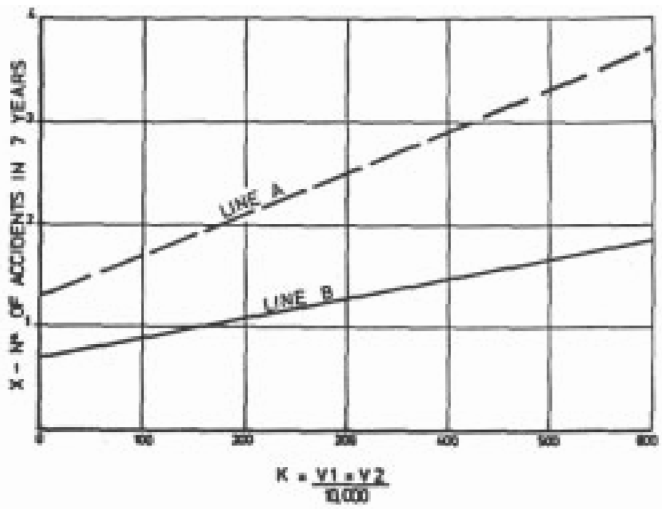


Figure 12.1

Crash Levels Above Which Treatment Should be Considered

Source: O'Brien (1976)

12.2 COST-EFFECTIVENESS

The studies reviewed generally did not provide sufficient data to provide true comparative cost-effective indications. However, by using the ADI (1981) figures an indicative BCR of 5:1 can be determined. This may be compared to the range 3:1 to 6:1 obtained for overtaking lanes which have a related function.

12.3 FURTHER RESEARCH

Present research into the relative safety of intersections and junctions in rural areas is limited and provides considerable scope for future research.

13.0 RAILWAY CROSSING TREATMENTS

13.1 INTRODUCTION

Crashes of any type at at-grade rail crossings are rare (Nicholas Clark & Associates 1984; Roy Jorgensen Associates Inc. 1978). Crashes involving a train are even rarer: two thirds of crashes investigated in a United States study of 7,500 at-grade crossings did not involve a train (Roy Jorgensen Associates Inc. 1978). It was also reported that the mean number of annual crashes involving trains was 0.15 per crossing.

Sinclair and Knight (1973) note that the design of approaches to railway crossings and the roadside structures in the vicinity is often poor. Tight radius curves are common. However, as noted by Sinclair and Knight a disproportionate number of fatality crashes occur at railway level crossings. Hence, while the frequency of this type of crash is generally low the severity is relatively high.

Considerable research has been undertaken on the factors affecting crashes and the impact of treatments at rail-road crossings. The most comprehensive international reference is a report prepared by the Texas Transportation Institute for the Federal Highway Administration - (Pinnell et al 1982). Middleton (1978) provides a comprehensive assessment of the safety and other aspects of control devices at railway level crossings in Queensland based on a theoretical modelling of safety (and road user) costs and of certain site characteristics.

13.2 INFLUENTIAL FACTORS

Influential factors on the frequency of railway crossing crashes based on Pinnell et al (1982) and Middleton (1978) are:

- driver knowledge and attitude: particularly correct speed perception, and crossing familiarity (Pinnell et al 1982);

- driver behaviour such as looking to see that a crossing is clear; a high proportion of drivers do not appear to look (Pinnell et al 1982);

- angle of skew (angle of approach): at crossings with angle of skew less than 70° there appear to be fewer vehicle-train and more non-train involved crashes than at crossings with angles greater than 70°. On average, the former crash types are more severe and costly than the latter. Hence, crash cost appears independent of angle of skew (Middleton 1978);

- the type of crossing and the relationship to road alignment: uncontrolled crossings on S-bends have from 2 to 10 times more crashes than on crossings with straight approaches;

- road and rail volumes;

- traffic and train speed;

- available sight distance;

- existing warning level;

- surface condition.

Middleton (1978) showed that there are major differences in the safety characteristics of rural and urban rail-crossings. He considered that this may be due to the combined effect of many factors including train and vehicle speeds.

13.3 TREATMENTS

Treatments can be categorised as:

passive crossing control devices;
active crossing control devices;
crossing elimination or abandonment; and

on-train devices which are not considered here (refer to Pinnell et al 1982 for details).

A summary of the applicability of treatments, costs and likely crash reductions are summarised in Table 13.1.

13.4 COST-EFFECTIVENESS RATING

Table 13.2 shows the cost-effectiveness ratings of the various treatments shown in Table 13.1 using a % reduction for $10000 cost rating method.

The results indicate the following:

For Passive Treatments

That improved advance warning signs, on intuitive grounds, appear to offer a high safety cost-effective rating primarily because of their low cost.

Stop signs are of little safety value except at sites with unfavourable physical characteristics. Road user cost increases would usually outweight potential safety benefits.

Rumble strips are of uncertain value and may increase road user costs through speed cycle changes and decrease safety if drivers attempt to avoid them.

For Active Treatments

Flashing lights alone offer the highest safety cost-effective rating followed by boom gates.

The combination of boom gates with flashing lights although safer than the boom gates or flashing lights alone, is not as cost effective.

Table 13.1

Summary of Applicability of Rail Crossing Treatments, Costs and Likely Crash Reduction

Type of Treatment/Goal	Conditions for Installation (for Urban and Rural if Available or Known) Urban	Rural	Cost (1987)	Likely Crash Reduction
Passive				
Stop signs (Middleton 1978)	Not appropriate since crash saving minor and overwhelmed by road user cost increase.	Need should be based on visibility and other criteria. Effective in reducing crash costs but road user costs increases would usually out-weigh safety benefit.	Negligible	Little data available
Improved advance warning signing/to increase driver awareness of the need to look for trains	Use of appropriate advance warning signs is suggested by Pinnell et al (1982). However benefits not confirmed. Data collection costs would exceed costs of deploying the advance sign.		Negligible	Minor
Rumble strips/to slow drivers & increase driver awareness	Benefits are inconclusive as driver "surprise" or "avoidance" may increase other crash type. If applied widely drivers may become accustomed.		Negligible	Uncertain
Active				
Flashing lights/to increase awareness	Warranted at higher exposure indices (VT): where V = AADT and T = Trains/week . when approach angle less than 70^o warranted when VT GE 90,000 . when approach angle greater than 70^o warranted when greater than/equal to 170,000		$35,000 in 1987 prices (Based on Nicholas Clark & Associates 1984)	64% reduction in all crashes with passive "before" condition. (Pinnell et al 1982 ADI 1981), suggests similar results with higher reduction in fatality crashes: up to 90%.

Table 13.1

Cont'd.

Boom gates/to physically prevent crossing access	Middleton (1978) found that boom gates were the most effective means of reducing crashes at urban rail crossings. Boom gates are warranted when: . approach angle less than 70°; for VT greater than/equal to 120000 . approach angle greater than 70°; for VT greater than/equal to 300000	$50,000	70% reduction if relative to average conditions. (Pinnell et al 1982) up to 90% reduction in fatality and injury crashes.
Flashing lights with boom gates/to increase awareness and physically prevent access	. See notes above.	$80,000	88% reduction if relative to passive control. 64% reduction relative to flashing lights.
<u>Crossing relocation, consolidation, abandonment</u>			
To remove conflict e.g. grade separation	Decision to relocate or abandon relates mainly to other factors.	Very high	No data relative to flashing lights but minor, relation to cost.

Table 13.2

Safety Cost-Effectiveness Rating of Treatments

Treatment	% Crash Reduction/ Average Cost $10,000
Passive	
Stop signs	Minor
Improved advance warning signing	High
Rumble strips	Uncertain
Active	
Flashing lights	18
Boom gates	14
Flashing lights with boom gates (relative to passive control)	11
Flashing lights with boom gates (relative to flashing lights)	8
Crossing Relocation etc.	Very low

Crossing Relocation

Would not normally be warranted on safety grounds because of the high cost.

Utilising the standardisation methods of Section 2.0 gives indicative BCR's as follows:

Flashing lights	0.6
Boom gates	0.5
Both	0.4

These are for average rural crossings. Where flows of trains or road vehicles are higher than average, the BCR's could exceed 1.

13.5 FURTHER RESEARCH

Further research on the safety and cost-effectiveness of rail-road crossings is considered to be required with respect to the actual number of crashes occurring at such locations. This would involve analysis of actual crash records.

14.0 NEW ROAD CONSTRUCTION

14.1 INTRODUCTION

New road construction is defined as complete new construction or major road upgrading such as duplication.

Research from the United States indicates that driving on rural roads is up to six times more dangerous than driving on urban roads (Balcar, 1981). Rural road conditions in Australia vary widely and range from lightly trafficked two lane unsealed roads as narrow as four metres in width to high speed highways consisting of two or more lanes. A large number of crashes on rural roads involve vehicles travelling in opposite directions and these crashes, particularly "head-on" crashes, tend to be of high severity.

The intuitive case for road duplication and new road construction to effect safety improvements appears strong. Studies of crash rates on roads of different characteristics show that crash reductions are potentially able to be derived from new road construction (Balcar, 1981; Searles, 1986). Table 14.1 presents data published by Moskowitz (1961) which illustrates this. Cox et al (1979) examined the crash rates following the construction of freeway standards in Sydney's outer urban areas. The conclusions are reproduced below:

> "The first section of the study consisted of a series of 'before/after' comparisons of casualty crash rates based on casualty crashes per 100 million vehicles kilometres. The 'before' casualty crash rates on three highways were compared with casualty crash rates 'after' the freeways were opened on: (i) the parallel freeways, (ii) the parallel freeways and highways combined and; (iii) the highways.
>
> Analysis of variance was used to test the significance of the results.

Table 14.1

Accident Rates on Rural Highways Related to Design Standards

	2-LANE		3-LANE		4-LANE UNDIVIDED		4-LANE* DIVIDED		DIVIDED,† CONTROLLED ACCESS		FREEWAY‡		TOTAL§	
Miles	10,450		45		167		210		794		430		12,330	
Million Vehicle-Miles	8,358		232		976		1,234		3,543		3,052		17,525	
Average Daily Traffic	2,191		14,239		15,997		16,130		12,224		19,449		3,894	
	No.	Rate¶	No.	Rate	No.	Rate	No.	Rate	No.	Rate	No.	Rate	No.	R
Total Reported Accidents	19,899	2.38	597	2.57	3,995	4.09	,591	2.91	6,011	1.69	3,066	1.00	37,640	2
Single-Vehicle Accidents	7,058	0.84	113	0.49	367	0.38	489	0.40	1,532	0.43	862	0.28	10,481	0
Collisions Between 2 or More Vehicles:														
(a) Between Intersections:														
(1) Head-on	3,152	0.42	67	0.29	197	0.20	68	0.06	216	0.06	139	0.045	4,220	0.
(2) Non-Head-on	3,675	0.44	203	0.87	966	0.99	953	0.77	2,099	0.59	1,926	0.63	9,941	0.
(b) At Intersections	5,654	0.68	214	0.92	2,465	2.52	,081	1.69	2,164	0.61	139£	0.045	12,998	0
Total Excluding Intersection Accidents		1.70		1.65		1.57		1.22		1.08		0.95		1

Source : Moskowitz (1961)

NOTES:

* 4-lane divided roads have a median separating opposing traffic but roadside access is uncontrolled.

† Divided controlled-access roads are nearly all 4-lanes with a few miles of 6-lane. Opposing traffic is separated and there is no access except at intersections. However, intersections at grade are frequent and traffic enters and exits at large angles, approximating 90°. All State highways except freeways require approaching traffic on cross roads to stop before entering or crossing the State highway, unless the intersection is controlled by traffic signals and the light is green.

‡ Freeways are defined in the text.

§ Total is different from sum of all columns because it includes some highways that are not classified in one of the columns.

¶ Rate is number of accidents per million vehicle-miles.

£ Accidents at ramps.

The overall casualty crash rate on the highways 'before' was 59 per 100 million vehicle kilometres compared to 12 on the freeway alone in the 'after' period and 36 on the freeways and highways combined. The difference in crash rates for the highways 'before' compared with highways 'after' was not significant, suggesting that the reduction in casualty crash rates was principally due to the opening of the freeways to traffic.

The second section of the study analysed casualty and non-casualty two-way rates on the freeways and on comparable sections of highway standard road along the same traffic routes. Analysis of variance was used to test the significance of the results.

Significantly lower crash rates were experienced on the freeways (confidence level not reported). The overall crash rate was 31 per 100 million vehicle kilometres on the four freeways studied and 167 on the comparable road sections.

The construction of outer urban and rural freeways in Sydney has therefore produced a significant benefit by way of lower crash rates."

No other such 'before' and 'after' studies were identified. Other references appear to base data on comparisons of the crash rates on roads of different geometric standards as presented in Table 14.1.

The safety benefits of urban freeways are well demonstrated and relate mainly to the degree of the type of access control. It has been suggested that access control has the potential of reducing crash rates by up to 60% and that even partial control of access will substantially reduce crash rates (Stover et al 1982). Under normal conditions significant access control can usually only be obtained in conjunction with a new road. Searles (1986) reports that, in the United States, casualty and fatality rates on conventional roads (based on distance travelled) are typically three to five times that on freeways. This result is borne out by Stover et al (1982).

However, the case for new rural roads is less clear. The majority of the research studies on this topic have been carried out in America and therefore have probably included divided highways, in and near urban areas, which probably have a high level of roadside activity. Possibly a study involving only rural divided highways, such as are common in Australia, could more easily demonstrate the safety benefits of dividing highways. However the low-volume nature of many Australian rural roads could make data collection for such a study expensive if the results were to be widely applicable.

In addition on lightly trafficked roads (say, lower than 2000 vehicles per day), the evidence suggests that driver and vehicle characteristics are the most significant factors in crashes (Corner 1983). Consequently, the safety benefits and cost-effectiveness of new road construction for lightly trafficked roads will usually be significantly less than that applying to higher volume, higher standard roads.

14.2 INFLUENTIAL FACTORS

The following factors appear to be influential on the scale and type of safety benefits to be derived from new road construction:

- roadside environment characteristics of the existing road, such as lane-use, degree of at-grade access, volume and associated width or number of lanes provided;

- standard of the road to be provided and the characteristics of the roadside environment: safety benefits will be larger if the new road is designed to consistent standards using McLean's (1983) concept of speed environment to minimise 'surprises'.

There is some evidence that there are diminishing safety returns in facility provision: the safety benefits of going from a 1 to 2 lane and a 2 to 4 lane road are likely to be greater than that for going from, for example, a 4 to 6 lane road.

14.3 COST-EFFECTIVENESS RATING

It is possible to calculate the crash reductions between roads of different design standards as are presented in Table 14.1 and compare them to incremental capital (and maintenance costs) to calculate a Safety Cost-Effectiveness Rating. However, the data in Table 14.1 are old and do not necessarily relate to the benefits derived by new roads compared to an existing situation.

However, as discussed above there is a strong case for access control as can be derived from a new highway, or freeway where crash reductions of between 50-80% can be expected. These roads would normally be heavily trafficked.

A typical situation would be the upgrading of a 2-lane road to a 4-lane divided road possibly with some access control or the construction of a new 2-lane road with access control to bypass existing development. For the latter case an average per kilometre cost would be of the order of \$0.7 million without land acquisition. The safety cost-effective rating would be of the order of:

$$\frac{(60\% \times 10{,}000)}{(\quad 700{,}000 \quad)} = 0.9$$

Converting this to a BCR gives approximately 0.1:1.

14.4 FURTHER RESEARCH

Further research is required to investigate crash reductions that accrue from the implementation of different classes of new roads. Research from the United States indicates that total discounted safety benefits alone are estimated to recoup over half of the total discounted cost of the interstate highway system (Searles, 1986).

Consequently, the case for this type of research is strong. Such research on low volume rural roads may not be cost-effective however.

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APPENDIX A

ABBREVIATIONS

ABBREVIATIONS

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
ACT	Australian Capital Territory
ARRB	Australian Road Research Board
ASTM	Australian Standards and Testing Methods
BCR	Benefit Cost Ratio
DMR	Department of Main Roads (N.S.W.)
DOT	(Federal) Department of Transport
DTP	Department of Transport (UK)
Fat.	Fatality
FHWA	Federal Highway Administration (US)
FORS	Federal Office of Road Safety
ft	feet
FYRR	First Year Rate of Return
g	gravity
GE	Greater Than or Equal To
GT	Greater Than
HRB	Highway Research Board (US) (predecessor to TRB)
HVOSM	Highway-Vehicle-Object Simulation Model
IRR	Internal Rate of Return
Inj.	Injury
IRRD	International Road Research Documentation
km	kilometre
kph	kilometres per hour
LASORS	Literature Analysis System on Road Safety
LE	Less Than or Equal To
LT	Less Than
m	metre
MEV	Million Entering Vehicles
mm	milimetre
mph	miles per hour
MU	coefficient of friction
MVK	Million Vehicle Kilometres
MVM	Million Vehicle Miles
NAASRA	National Association of Australian State Road Authorities
NCHRP	National Cooperative Highway Research Program (US)
NRMA	National Roads and Motorists' Association
NSW	New South Wales
NPB	Net Present Benefit
NPC	Net Present Cost
NPV	Net Present Value
NT	Northern Territory
PDO	Property Damage Only
PSDN	Predicted Stopping Distance Number
Qld	Queensland
RACV	Royal Automobile Club Victoria
ROR	Run Off Road
RRL	Road Research Laboratory (UK) (predecessor to TRRL)
RRPM's	Raised Reflective Pavement Markers
RTA	Road Traffic Authority (Victoria)
SA	South Australia
Tas	Tasmania
TRB	Transportation Research Board (US)

TRRL	Transport and Road Research Laboratory (UK)
SI	Severity Index
SSD	Stopping-Sight Distance
UK	United Kingdom
US(A)	United States
Vic	Victoria
WA	Western Australia
yr	year