2 Basic Design Criteria

2.1 Design Controls

2.1.1 General

The main controls influencing the design standards used for state highway roading projects are:

(a) Finance

Funding for state highway roading work, new construction or improvements to existing highways, is dependent on the economic benefit derived and expenditure has to be justified in terms of the resulting value.

The design standard adopted for a particular work depends on the level of development of the state highway roading network in the particular area. When the network is substantially adequate in terms of traffic demand, and suitable finance is available, projects will normally be directed at improving operational safety and efficiency and higher geometric standards are appropriate. When the network is inadequate in terms of traffic demand, and finance is not readily available, lower geometric standards may sometimes be adopted on selected sections of road, provided an adequate operational improvement can be obtained.

(b) Terrain

Research has shown that terrain is one of the main factors that affect driver behaviour. It is therefore an important factor in the selection of design parameters for a roading project.

Terrain also has a significant effect on the costs of achieving a high standard road alignment. In flat terrain high standard roads can be achieved at practically no additional cost while in steep mountainous country costs escalate rapidly if high standards are used.

2.1.2 Traffic Volume

The volume of traffic to be carried by a road is the loading the designer must satisfy. Capacity analysis, level of service and design traffic volumes are used to determine the number of traffic lanes required.

Capacity analysis is described in the *Austroads Guide to Traffic Engineering Practice - Part 2: Roadway Capacity* and the *Highway Capacity Manual (TRB 1985).* Design traffic volumes are estimated for a future year, usually 25 years ahead, and the road designed to carry this demand at a predetermined level of service, usually C.

2.1.3 Traffic Composition

The effect of heavy vehicles in the traffic stream is to lower the level of service provided by the road because:

- a heavy vehicle takes up more road space than a car so it is equivalent to more than one car in traffic volume terms, and
- the disparity in speeds between light and heavy vehicles leads to increased queuing and overtaking requirements.

The proportion of heavy commercial vehicles in the traffic stream influences the structural design of the road pavement. It also influences aspects of geometric design because of the disparity of speeds on grades and their greater width, eg. heavy vehicles need climbing lanes as well as pavement widening on curves and turning roadways.

2.1.4 Safety

Safety is a major goal of road design. The theme *'of enabling drivers to perceive hazards in time to take the appropriate action through the use of geometric parameters consistent with the likely speed of traffic operation'* runs throughout this manual. Vehicles do get out of control, hence items such as traversable batter slopes, guard rails, breakaway light columns and sign supports are desirable features of what has been described as a forgiving roadside.

2.1.5 Environment

Environmental factors need to be considered in major public works such as road construction and, at the level of development of New Zealand, these are an essential part of the road design process.

A road is just one element in the environment. Environment in this sense refers to the total social and natural environment. A road should therefore be located and detailed so as to compliment the environment and to serve the surrounding communities rather than to harm them. For example, a valuable resource such as an adjacent area of high quality agricultural land, may merit preservation in its own right and could restrict the land available for expansion of the road right of way.

2.1.6 Speed

Speed is the most important parameter in roading design. Speed is used to select geometric design features such as alignment and cross section elements. It must reflect actual operating speeds on the road to ensure the safe and efficient movement of traffic.

A good design combines all geometric elements into one harmonious whole consistent with the speed environment so that drivers will be encouraged to maintain a reasonably uniform speed over as long a length of road as possible.

2.1.7 Aesthetics

Aesthetics is concerned with the view of the road from the users perspective as well as the view of the surrounding area from the road. This is particularly important for roads in scenic areas.

2.1.8 Energy Use

The total road task makes considerable use of liquid fuels and other products derived from crude oil. Grades steeper than about 5% causes a greater consumption of fuel by heavy vehicles in the uphill direction than what they save in the downhill direction.

At the present time the flattening of grades can rarely be justified on the basis of energy saving alone. The greatest changes in energy consumption related to transport come from using the most appropriate mode for long distance freight haulage, and from land use distributions in urban areas.

2.1.9 Stage Construction

Where land use is changing and traffic is growing no road can ever be regarded as final and there will always be requirements for future road improvement or modification. Where it is obvious that medium term requirements are different to the best short term design for a particular road, it is often possible to modify the design slightly to provide better options for the future. While this might tie up some funds and prevent their use on other current projects the effect can be much less than if a longer term design was adopted in the first instance.

One area where this approach is relevant is the high standard low volume road. In these cases extra care must also be taken not to commit large amounts of current funds for very long term options.

2.2 Road Classification

2.2.1 General

A roading network comprises various road types which facilitate vehicular travel between points of trip origin and destination as well as providing access to property.

Road classification is the grouping of roads into orderly systems according to the type and degree of service they provide to the public. When a road system is properly classified the characteristics of each road are readily understood.

Many different classification systems have been developed and used for various purposes. A road classification based on land service, traffic service and traffic use is however the most useful because it is adaptable for both planning and design purposes.

The information required to identify the service classification normally includes traffic volume, traffic flow characteristics, distribution of vehicle types, speed characteristics of different vehicles, adjacent land use, and the degree of land service provided. Figure 2.1 illustrates the relationship of the traffic operation and land access functional requirements for various types of road.

2.2.2 Functional Classification

Functional road classifications range from major arterials , ie. motorways, to local access roads and Transit has adopted a system which caters for both rural and urban roads. Road classifications are not always clear cut however because state highways usually have some degree of local road importance, eg. a local road which forms part of a state highway may have uses related a higher hierarchical level superimposed on its local significance.

High functional class rural roads generally need to cater for a greater proportion of longer length journeys. Higher design standards should be applied to these roads to ensure the quality of service provided is more appropriate to longer duration trips. However, care must be taken not to attach too much importance to functional class alone where traffic volumes are low.

2.2.3 Road Hierarchy

A roading hierarchy is an interrelated and consistent classification of roads that accommodates the impacts of traffic operation and land use.

A functional road hierarchy establishes a logical integrated network in which roads of a similar functional classification are:

- provided with the same general level of traffic service with regard to trip purpose, traffic composition, capacity and operational speed,
- designed, constructed and maintained to the same general level of structure with regard to alignment, cross section, pavement strength and access control, and
- provided with similar operational treatment regarding signposting, pavement marking, intersection layout and provision for pedestrian crossing.

The implementation of a roading hierarchy will:

- determine the appropriateness of land use activities in various localities,
- determine standards of vehicle access for properties and activities, and
- assist in determining appropriate environmental outcomes.

The road hierarchy classification system used by Transit New Zealand is:

(a) National Routes:

Motorways, expressways and major two-lane roads which:

- form a nationally important strategic roading network,
- are significant elements in the national economy,
- have the highest degree of access standard and control, and
- provide a high level of user service at all times.

(b) Primary (Regional) Arterials:

Major roads which:

- form strategic links between regions, or within regions and between districts,
- are significant elements in the regional economy, and
- have some access controls and standards for permitted activities which are determined mainly on the basis of strategic function and traffic volume.

(c) Secondary (District) Arterials:

Roads which:

- form strategic links within, or between, districts,
- are significant elements in the local economy, and
- often also serve as local roads.

Access standards for these roads are determined by the careful consideration of:

- form (the physical alignment of the road),
- function (the present and future role of the road), and
- traffic volumes.

(d) Collector Routes:

Locally preferred routes between, or within, areas of population or commercial activity which:

- complement district arterials but have property access as a higher priority, and
- have standards suitable for the safe operational requirements of the traffic volume on each section.

(e) Local Roads:

All other roads. The standards for these roads are those considered appropriate by the road controlling authority for local traffic operation and land access requirements.

	Local Road	Collector Road	Arterial Road	Expressway	Motorway
Traffic Function	traffic movement is the secondary consideration	traffic movement and land access are of equal importance	traffic movement is the primary consideration	traffic movement is the primary consideration	optimised traffic movement is the primary consideration
Land Access Function	land access is the primary consideration	land access and traffic movement are of equal importance	some access control	rigid access control	no access
Traffic Volume (AADT)	<1000	<5000	<12000	>8000	>8000
Flow Characteristics	interrupted flow	interrupted flow	uninterrupted flow except at intersections (signalised)	uninterrupted flow except at signalised intersections	free-flow (grade separated intersections)
Design Speed (km/h)	50 - 110	60 - 110	80 - 130	100 - 130	100 - 130
Average running speed in free flow conditions (km/h)	50 - 90	50 - 90	60 - 100	60 - 100	70 - 110
Vehicle type	cars, light / medium trucks, occasional heavy trucks	all types, up to 30% trucks in the 3 t to 5 t range	all types, up to 20% heavy trucks	all types, up to 20% heavy trucks	all types, up to 20% heavy trucks
Normally connect to:	locals, collectors	locals, collectors, arterials,	collectors, arterials, expressways, motorways	arterials, motorways	arterials, expressways, motorways

Table 2.1: Road Classification Characteristics

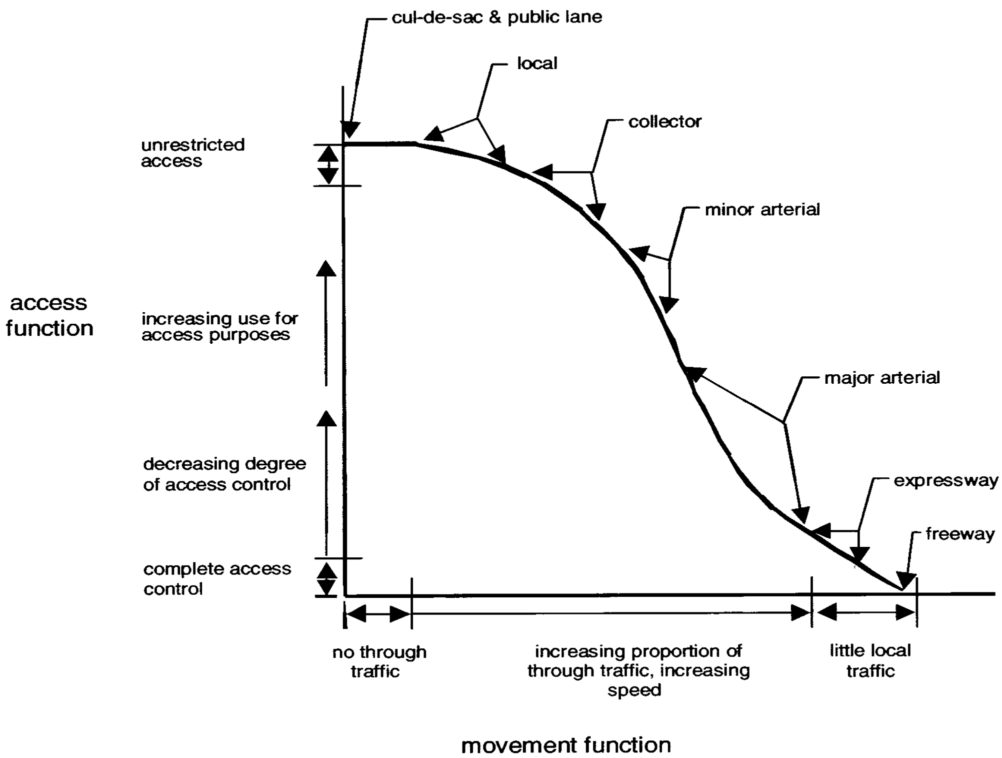


Figure 2.1: Road Service / Function Relationships

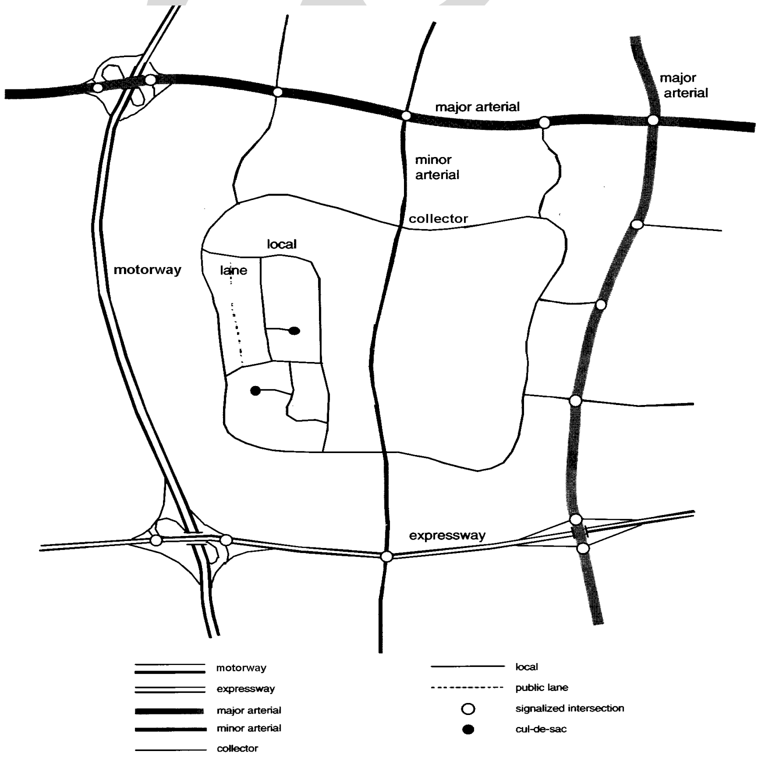


Figure 2.2: Road Classification and Hierarchy

2.3 Traffic Flow and Composition

2.3.1 Traffic Flow

The basic theory and characteristics of traffic flow are described in the Austroads *Guide to Traffic Engineering Practice - Part 1: Traffic Flow*, including:

- traffic streams and their elements,
- transverse distribution of vehicles in the traffic stream,
- longitudinal distribution of vehicles,
- regional distribution of vehicles,
- statistical distribution of traffic,
- gap acceptance,
- queuing and delay, and
- vehicle interactions in the traffic flow.

2.3.2 Traffic Volumes

Traffic volumes are generally expressed in terms of annual average daily traffic (AADT), average weekday, average weekend/holiday, average hour or average quarter hour volumes. Seasonal variations might be relevant, particularly on roads likely to have a significant proportion of recreational traffic. An understanding of the mix of car and truck traffic is needed to evaluate the need for climbing and passing lanes.

2.3.3 Traffic Volume Surveys

Traffic volume surveys are described in the Austroads *Guide To Traffic Engineering Practice - Part 3: Traffic Studies* and the Transit New Zealand publication *Traffic Counting Guidelines.*

The following methods should used to estimate traffic volumes from traffic counts:

(a) AADT

To estimate AADT from a sample count it is usually necessary to adjust the count data by a number of factors. Count data must be checked for consistency and reasonableness. If it is an axle pair count, ie. from a tube counter, it is corrected by applying an adjustment factor to convert it to vehicle counts.

Daily counts for less than a week are adjusted by applying day factors for the appropriate typical traffic pattern to derive weekly average daily traffic. Weekly average daily traffic figures are adjusted by applying the appropriate week factors to derive an AADT. If more than one week is counted the AADT should be determined for each week and then averaged.

The appropriate traffic pattern control group from Transit's traffic counting guideline can be used to determine day and week factors. Alternatively, these factors may be derived from a rigorous local traffic counting programme.

(b) Weekday or Weekend/Holiday Volumes

Weekday, Saturday and Sunday/holiday volumes may be calculated from AADT's by applying locally derived day factors or day factors from Transit's traffic counting guideline when local data is not available. Saturday and Sunday/holiday volumes must be averaged to give average weekend/holiday daily volumes.

(c) Hourly or Quarter Hourly Directional Volumes

Short time period traffic volumes can be obtained from traffic counter records or manual counts. Traffic counter records for AADT also usually contain 15 minute and/or hourly traffic volumes. Factors from Transit's traffic counting guideline can also be applied to daily traffic counter records to obtain estimates of 15 minute and hourly traffic volumes.

(d) Intersection Volumes

At intersections a manual count of turning movements should be made on a representative day of the week. A minimum count of one-hour, subdivided into quarter-hour periods, should be taken for each of the following periods:

- Morning peak,
- Mid-morning,
- Mid-day peak,
- Mid afternoon,
- Evening peak, and
- any other relevant peak period.

2.3.4 Equivalent Car Unit

Wherever possible observed traffic data should be used to determine axle pair adjustment factors. To establish the equivalent car unit (ECU), in the absence of such data, the factors in Table 2.2 may be used to convert axle pairs to ECU's.

Road Category	Axle Pair Adjustment Factor
Urban	0.91
Rural	0.83

Source: Transfund New Zealand Project Evaluation Manual

Table 2.2: Axle Pair Adjustment Factor (E)

2.3.5 Traffic Growth Rate

Traffic growth is defined with an arithmetic growth rate rather than a geometric growth rate and is expressed as a percentage of the current traffic volume.

Wherever possible actual traffic count data from the site, or a suitable adjacent site (s), should be used to determine the current traffic growth rate. At least four counts in the last six years, or seven or more counts in the last ten years, are needed to estimate a traffic growth rate and a linear regression used to best fit the data. The results must be checked for reasonableness with traffic count data from nearby sites.

To estimate a combined traffic growth rate for several sites the growth rate for each site must be computed and then a weighted average calculated.

2.3.6 Roadway Capacity

Roadway capacity theory and its practical application is described in the Austroads *Guide to Traffic Engineering Practice - Part 2: Roadway Capacity*. This guide reviews material from the 1985 US *Highway Capacity Manual* and includes more recent Australian work on roadway capacity.

Capacity is defined as the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under the prevailing roadway, traffic and control conditions. The following points should be noted with respect to this definition:

- The time period used in most capacity analyses should be 15 minutes. This is normally considered to be the shortest interval during which stable flow exists.
- The prevailing roadway, traffic and control conditions should be reasonably uniform for the section of roadway being analysed.

Roadway Conditions are the geometric characteristics of the road and includes the type of road, its development environment, the number of lanes in each direction, lane and shoulder widths, lateral clearances, design speed and horizontal and vertical alignments.

Traffic Conditions refers to the characteristics of the traffic stream using the road, including vehicle type and the lane and directional distribution of the traffic.

Control Conditions are the types and specific design of the control devices and traffic regulations applicable to the particular section of road.

2.3.7 Level of Service

Level of service is a qualitative measure which describes operational conditions within a traffic stream, and their perception by motorists and/or passengers. Level of service generally describes operational conditions in terms of factors such as speed and travel time, freedom to manoeuvre, traffic interruptions, comfort and convenience and safety.

Six levels of service have been designated, A to F. Level of service A represents the best operating condition, ie. free flow, and level of service F the worst, ie. forced or break-down flow. Each level of service is described in more detail in the following paragraphs.

(a) Level of Service A

This is a condition of free flow in which individual drivers are virtually unaffected by the presence of others in the traffic stream. They have freedom to select desired speeds, the ability to manoeuvre within the traffic stream is extremely high and the general level of comfort and convenience is excellent.

(b) Level of Service B

This is within the zone of stable flow and drivers still have reasonable freedom to select their desired speed and to manoeuvre within the traffic stream, although the general level of comfort and convenience is a little less than level of service A.

(c) Level of Service C

This is also in the zone of stable flow but most drivers are restricted to some extent in their freedom to select their desired speed and to manoeuvre within the traffic stream. The general level of comfort and convenience declines noticeably at this level.

(d) Level of Service D

This is close to the limit of stable flow and approaching unstable flow. All drivers are severely restricted in their freedom to select their desired speed and to manoeuvre within the traffic stream. The general level of comfort and convenience is poor and small increases in traffic flow will generally cause operational problems.

(e) Level of Service E

This occurs when traffic volumes are at or close to capacity and there is virtually no freedom to select desired speeds or to manoeuvre within the traffic stream. Traffic flow is unstable and minor disturbances within the traffic stream will cause break-downs.

(f) Level of Service F

This is a forced flow condition when the amount of traffic approaching the point under consideration exceeds that which can pass it, flow break-down occurs and queuing and delays result.

The level of service concept can be used as a basis of capacity and operational analysis for all types of road facilities. In this manual it is used mainly for the analysis of two lane roads, multi-lane roads motorways.

Various parameters can be used to define levels of service for each type of facility but, for practical purposes, the parameters that best describe quality of operation have been selected as measures of effectiveness. These measures are summarised in Table 2.3.

Type of Facility	Measure of Effectiveness
Two-Lane Roads	Percent time delayed - percent Average travel speed - km/h
Multi-Lane Roads	Density - ecu/km/lane
Motorways:	
Basic Segments	Average travel speed - km/h
Weaving Areas	Density - ecu/km/lane
Ramp Junctions	Flow rates - ecu/h

Source. TRB (1985), Table 1.2

Table 2.3: Measures of Effectiveness for Defining Levels of Service

NOTE: ***ecu = equivalent car units, as defined in Section 2.3.4***

2.3.8 Service Flow Rate

Service flow rate is defined as the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway, during a given time period and under the prevailing roadway, traffic and control conditions while maintaining a designated level of service. In a similar to capacity, service flow rate is generally measured over a 15 minutes time period.

Five service flow rates are defined for each type of road facility, ie. A to E. Break-down flow occurs at level of service F so a meaningful service flow rate cannot be specified.

Each service flow rate specified is a maximum so they effectively define the boundaries between each level of service.

2.3.9 Degree of Saturation

Degree of saturation is the term used to describe the operational characteristics of a signalised intersection. It is a ratio but is often expressed as a percentage.

Saturation flow is defined as the maximum rate of flow of vehicles across the limit line at a signalised intersection approach, during the effective green time and while there is a continuous queue of vehicles waiting to move during that time. It can be applied either to either individual lanes or to the total approach width and is equivalent to the capacity of the individual lane or approach.

The degree of saturation of a signalised approach is defined as the ratio of the arrival flow, or demand, to the capacity of the approach during the same period. The normal range for the degree of saturation is from almost zero at very low traffic flows, to nearly 1 for saturated flows **or** at capacity. A degree of saturation greater than 1 indicates an oversaturated condition where long queues will build up on that approach. In general the lower the degree of saturation the better the quality of traffic service.

A starting point in the analysis of capacity, level of service or degree of saturation is to select values that are applicable to ideal conditions. Correction or adjustment factors can then be applied to reflect the actual roadway, traffic and control conditions.

In general, an ideal condition is one where further improvements will not result in any increase in capacity or level of service, or decrease the degree of saturation. More specific details of ideal conditions are given in the Austroads *Guide to Traffic Engineering Practice - Part 2: Roadway Capacity.*

2.3.10 Roadway Conditions

Roadway conditions affecting capacity and level of service include:

- the type of facility and its development environment,
- traffic lane width,
- shoulder width and/or lateral clearance,
- design speed, and
- horizontal and vertical alignment.

Correction factors for these conditions are given in the Austroads *Guide to Traffic Engineering Practice - Part 2: Roadway Capacity.* These factor are based on US data and special care needs to be taken in their use for New Zealand conditions.

2.3.11 Terrain Conditions

For capacity analysis and road design purposes three general terrain classes are used. These are:

(a) Level

Any combination of grade, horizontal alignment and vertical alignment that will allow heavy vehicles to maintain approximately the same speeds as passenger cars.

(b) Rolling

Any combination of grade, horizontal alignment and vertical alignment that will cause heavy vehicle speeds to be reduced substantially below those of passenger cars, but will not cause them to operate at crawl speeds for any significant length of time.

(c) Mountainous

Any combination of grade, horizontal alignment and vertical alignment that will cause heavy vehicles to operate at crawl speeds for significant distances and/or at frequent intervals.

2.3.12 Vehicle Classifications

For capacity analysis and road design purposes road traffic is classified into three categories. These are:

(a) **Passenger cars:** Light vehicles and light delivery vans having no more than four single tyres.

(b) **Trucks:** Vehicles having more than four single tyres which are mainly used for the transport of goods or services.

(c) **Buses:** Vehicles having more than four single tyres which are primarily used for the transport of people. For capacity analysis purposes passenger cars towing caravans, boats and other similar recreation equipment should be included in this category.

These vehicle categories are generally consistent with current traffic counting classifications and take into account the level of precision of most capacity and related analyses.

2.3.13 Driver Characteristic Adjustments

The traffic stream characteristics described in Austroads *Guide to Traffic Engineering Practice - Part 2: Roadway Capacity* are representative of regular weekday commuter drivers and other regular users of a facility.

Capacity and/or levels of service may be reduced when the traffic stream contains a significant proportion of weekend, recreation, or mid-day (off-peak) drivers. Driver characteristic adjustment factors are given in the Austroads guide but the range of values is relatively wide and engineering judgement and/or local data should be used to select an appropriate value.

2.3.14 Control Conditions

The time available for individual traffic movements is a critical factor which affects the capacity, level of service and/or the degree of saturation of interrupted flow facilities.

Time controls at these types of facility include traffic signals, STOP and GIVE WAY signs, parking restrictions, turn restrictions, lane use controls etc.

The effects of these controls are described in the relevant sections of the Austroads *Guide to Traffic Engineering Practice - Part 2: Roadway Capacity.*

2.4 Vehicle Characteristics

2.4.1 General

The state highway system must be designed and constructed to readily accommodate all vehicles up to the legal maximum size. Where appropriate, allowance should also be made for the operation of special over-dimension vehicles which are peculiar to some areas.

Lane width, gradient, length of grades, curve radii, length of horizontal and vertical curves and clearances to overhead and lateral obstacles are all related to the overall size, length, width, height and mass of legal vehicles. Limits are imposed on these dimensions in order to:

- ensure that vehicles are compatible with the road space provided when geometric design standards similar to those of most overseas roading authorities are used,
- permit effective and efficient traffic control measures to be implemented, and
- safeguard the safety and interests of other road users.

2.4.2 Vehicle Tracking Requirements

Tracking curves show the behaviour of a single or a combination vehicle as it executes a defined turn. The LTSA *New Zealand On Road Tracking Curves* are the sole reference for on-road tracking diagrams but computer programs that generate tracking curves consistent with the LTSA standard may also be used to check designs.

Off-tracking is defined as the extent of the cut-in of the rear of the vehicle as it executes a turn.

NOTE: ***A tracking curve applies only to the specified vehicle and turn radius, the off-tracking will be different if either of these factors are varied.***

State highway intersections, roundabouts and other on-road facilities must be designed using tracking curves, to ensure that the appropriate design vehicle can manoeuvre safely without damaging other vehicles or road facilities.

2.4.3 Design Vehicles

A number of standard New Zealand design vehicles have been identified through various research projects, eg. a recent survey of New Zealand car fleets identified the 90 percentile car as one similar in length to a 1992 Mazda 626 sedan.

The design vehicles used by LTSA to produce their *New Zealand on Road Tracking Curves* are:

(a) Trucks:

- **Medium Rigid Truck:** At 8 m in length these trucks are larger than the vans and light trucks commonly used by couriers. They are used to transport small to medium sized consignments in local areas and are similar in length to a medium sized concrete truck, a rubbish truck, a moderate sized furniture moving truck and the trucks used to service convenience stores.
- **Large Rigid Truck:** 11 m in length and the largest rigid truck that can legally operate on New Zealand roads. These vehicles have the potential to carry larger and heavier loads, the flat deck versions usually being able to accommodate a standard 6 m (20 foot) International Standards Organisation (ISO) shipping container. They are commonly used to service large commercial, industrial and retail operations.
- **Semi-trailer:** The maximum overall length for this type of vehicle is 17 m. They are used to transport large consignments, including indivisible loads, overlong distances and are the only vehicles that can carry a standard 12 m (40 foot) ISO shipping container.
- **B-train:** 20 m is the maximum length for this type of vehicle and the maximum length allowable for any vehicle on New Zealand roads without a special over dimension permit. They can carry two 20 foot containers, one on each trailer, and are frequently used to transport bulk cargo on long haul routes.

(b) Buses:

- **Urban City Bus:** A two axle bus of the type frequently used in city and urban areas on time tabled routes. The dimensions are similar to a large rigid truck design vehicle so these tracking curves can also be used for an urban city bus.
- **Tour Coach:** 12.6 m is the maximum length permitted under current new Zealand heavy vehicle regulations. This type of bus is commonly used to transport tourists around the country.

(c) Cars:

- The 90 Percentile Car.

Other vehicles such as A-trains and Truck and Trailer combinations are not included in the list of design vehicles because their tracking requirements are very similar to those of B-trains. B-train tracking curves should therefore be used when designing for A-trains and Truck and Trailer combinations.

2.4.4 Design Clearances

Tracking curves show the theoretical path followed by the outermost points of the vehicle body and define the minimum physical space necessary for that vehicle to execute the manoeuvre. An additional clearance must be added each side of a tracking curve to allow for:

- variations introduced by driver unfamiliarity,
- steering/judgement errors,
- variations in design and actual vehicle dimensions, and
- to help protect items of street furniture such as street signs, lampposts, etc.

The desirable additional clearance for state highway intersection design is 1.0 m and it must never be less than 600 mm.

2.4.5 Minimum Design Radii

In New Zealand there is a legal requirement for all vehicles to be able to safely execute a 12.5 m radius turn.

Some vehicles can, however, turn on a significantly smaller radius without affecting their dynamic stability. Where this is possible, and considered reasonable, special tracking curves have been prepared. These tracking diagrams are only applicable to very low speed situations, ie. off-road storage sites, loading/unloading areas, etc. and must not be used for on-road design work.

For state highway intersection design the minimum design radii that can be used with the vehicles described in Section 2.4.3 are given in Table 2.4.

Design Vehicle	Design Radius (m)	
	Normal minimum	Absolute minimum
Passenger Car	10	8
Truck/Bus	15	12.5
Semi-Trailer	15	15
B-Train	20	15
NOTE: Absolute minimum design radii may only be used at very low design speeds, ie. ≤ 20 km/h.		

Table 2.4: Minimum Design Radii

A Semi-trailer and/or a B-train design vehicle should be used wherever a significant number of heavy commercial vehicles can normally be expected, eg. At intersections of state highways, at state highway and local arterial road intersections and at intersections within the local roading network of a large industrial area, etc.

In residential areas B-trains and Semi-trailers are not often seen but heavy commercial vehicles such as rubbish trucks, buses and furniture removal vans are common. The appropriate design vehicle for these areas is therefore a Large Rigid Truck or a Urban City Bus.

Special consideration must be given to the requirements of over-dimension vehicles on roads where these vehicles can be expected on a regular basis, eg. heavy haulage bypass routes, wharf access roads, etc, and special arrangements made to safely accommodate them. The local heavy haulage operators and Land Transport Safety Authority staff responsible for over-dimension permits must be consulted in these situations, to advise on the type, size and operational requirements of these vehicles.

2.5 Driver Characteristics

2.5.1 General

The most important driver characteristics are:

- a susceptibility to confusion from surprise,
- a tendency to act according to habit,
- the desire to take a *'natural'* movement path,
- the need for adequate reaction and decision making times, and
- an ability to select the correct action from several alternatives.

Drivers learn through experience that some events are likely to happen, or not happen. This expectancy is reinforced by the frequency an event does, or does not, happen. Examples of this type of behaviour are:

- travelling at dangerously small headways on the assumption that the vehicle in front will continue on at approximately its present speed, and
- not anticipating meeting a conflicting vehicle while driving through an intersection because that sort of problem has never been encountered at the site before.

To avoid surprises, and their possibly dangerous reactions, it is essential to provide drivers with a continuous flow of advice on the state of the road ahead, including information on:

- the road alignment,
- approaching decision points, and
- other traffic, vehicular or pedestrian activity which may affect, or be in conflict, with them.

This advice is mostly visual and can be provided by means such as road layout, signposting and pavement markings. The acceptance and subsequent treatment of this information depends largely on each driver's visual ability, reaction time and decision making processes.

2.5.2 Vision Characteristics

(a) General

Driving is basically a decision making process and it is very dependent on the driver’s perceptions and judgments.

It is commonly accepted that between 85% and 90% of the perceptions involved in the driving task are of a visual nature, ie. driving and visual perception are very closely related. The most important factors in road design are therefore:

- the ability of a driver to perceive things associated with the driving task
- the order in which a driver sees these things, and
- the time that they have to respond to what they see.

The average normal human eye can detect visual stimuli over approximately 180° in the horizontal plane and 130° in the vertical plane. It can also resolve 1 minute of arc which is equivalent to observing a 90 mm high object at a distance of 300 m.

Detailed, or foveal vision, is limited to about one thousandth of the visual field at any one moment and this represents a 2° cone of interest from the lens of the eye. The peripheral, or remaining, field of vision diminishes as the eye concentrates on detail. Also, the time taken in fixation, blinking and focussing can reduce overall vision by approximately 20%.

The ability of the eye to accurately judge distance between it and object is limited to approximately 100 m. Beyond that the lines of sight are almost parallel and the estimation of distance becomes dependent on comparisons and past experiences.

An observer can estimate the real size of objects at distances up to about 400 m from them. Beyond this distance objects begin to look smaller than they should, eg. a car at a 550 m distance is equivalent to a pin head at a distance of 450 mm from an observer’s eye.

A stationary observer has the ability to estimate the size of objects 400 m away. An overtaking driver travelling on a straight road at 100 km/h is however expected to be able to discern an oncoming vehicle and gauge its speed when it is at least 410 m away, even though only its roof may be visible above a crest.

On a long straight the inability of a driver's eye's to accurately determine whether a distant object is stationary, travelling towards or away, and at what speed, is due to a lack of visual comparison with other objects, both mobile and stationary. However, on a curving road an oncoming vehicle appears to travel horizontally across a driver's field of vision and comparison roadside objects and other traffic give a visual appreciation of position, speed and direction of travel.

(b) Vision at Speed

As speed increases the eye's behaviour will affect driver performance in the following manner:

- concentration decreases,
- the field of concentration recedes,
- peripheral vision diminishes,
- foreground detail begins to fade, and
- space perception becomes impaired.

It is desirable that a driver's view be directed along the line of the road to ensure that extraneous detail and distractions do not divert their attention away from the road. The cones of vision shown in Figure 2.3 indicate a driver's plan view at various travel speeds.

To ensure a particular aspect of the scene ahead is able to be seen by drivers, eg. an obstruction intruding onto the carriageway or a warning sign, the road must be designed so these types of feature will lie within the drivers cone of vision for the anticipated travel speed.

(c) Night Vision

A driver's vision is limited at night. Focal distance is limited to about 150 m, ie. the distance illuminated by a vehicle’s headlights, peripheral vision is reduced to almost zero, and detailed, or foveal, vision is restricted to the arc of light ahead.

Night vision is further reduced by headlight glare from oncoming traffic. This may be minimised on divided dual carriageways by the construction of glare screens or by providing a median. To reduce glare to a tolerable level a median may need a raised centreline section or, if depressed, be sufficiently wide, ie. at least 25 m.

(d) Wet Weather Vision

Figure 2.4 shows a comparison of design stopping and intermediate sight distances with the visibility available in a 25 mm/h intensity rainfall. In these conditions the minimum vision requirements for overtaking are not attainable at speeds in excess of 100 km/h.

There are no specific design guidelines on how to overcome wet weather visibility deficiencies but the design of road surface drainage should minimise the problem of vehicle generated spray. Certain combinations of grade and crossfall can create surface water ponding and special treatment of one, or both may be required to minimise this problem.

2.5.3 Reaction Time

Reaction time is the time for a driver to perceive and react to a particular stimulus and to take appropriate action. It depends on the complexity of the decision or task involved and includes the times taken to:

- perceive and identify an emergency situaton,
- judge what reaction is required, and
- activate the reaction process.

Reaction time is not a constant and varies according to a driver's:

- age,
- physical condition (fatigue, drugs and alcohol increase reaction time), and
- preparedness (presenting drivers with situations which match their expectation will reduce reaction time and vice versa).

Research has shown that most un-alerted drivers can react simply to a clear stimulus in less than 2.5 seconds, in an urgent situation. This represents an upper, possibly the 85th percentile, value for normal drivers and is close to the mean for degraded drivers. The variance of the distribution of reaction times is, however, very high and values ranging from 1 to 7 seconds have been recorded. One reason for the large variability is that reaction time depends on a driver's level of alertness at the time and anticipation or pre-signalling of an event, the absence of uncertainty on multiple choices, and the familiarity with the task can each lower reaction time. A reaction time of 2.5 seconds is a commonly adopted reaction time value for use in road design, although a number of European countries use a value of 2.0 seconds.

As people age, they experience decreasing physical and mental capabilities and become more susceptible to injury and shock. Human functions subject to deterioration due to ageing include:

- visual acuity,
- attention capacity,
- reaction time, and
- contrast sensitivity.

As a group, older drivers do not currently represent a major road safety problem in most Western societies when compared with other age groups. However, older drivers are involved in significantly more serious injury and casualty crashes per kilometre travelled and, because the proportion of older people in Australasia is expected to roughly double over the next 40 years, older drivers are likely to become a more significant problem in the years ahead.

Recent research indicates that a number of road design elements may be associated with older driver crashes in Australasia. In particular, it was concluded that improvements to intersection sight distances, provision for separate turn phases at traffic signals, more conspicuous traffic signal lanterns and more clearly defined vehicle paths have the potential to reduce crash and injury risk for older drivers. The research also contained a detailed description of measures that should be implemented immediately to increase the safety of older road users, including a recommendation that a minimum reaction time of 2.5 seconds be used at intersections. For mid-block sections it recommended a desirable minimum reaction time of 2.5 seconds and an absolute minimum of 2.0 seconds.

Two reaction times may be used in state highway road design work and these are:

(a) 2.5 seconds: For normal use in all situations. It must be used when design speeds are greater than 70 km/h because in these situations drivers are likely to be travelling in free speed conditions and are normally not so alert or prepared for the unexpected.

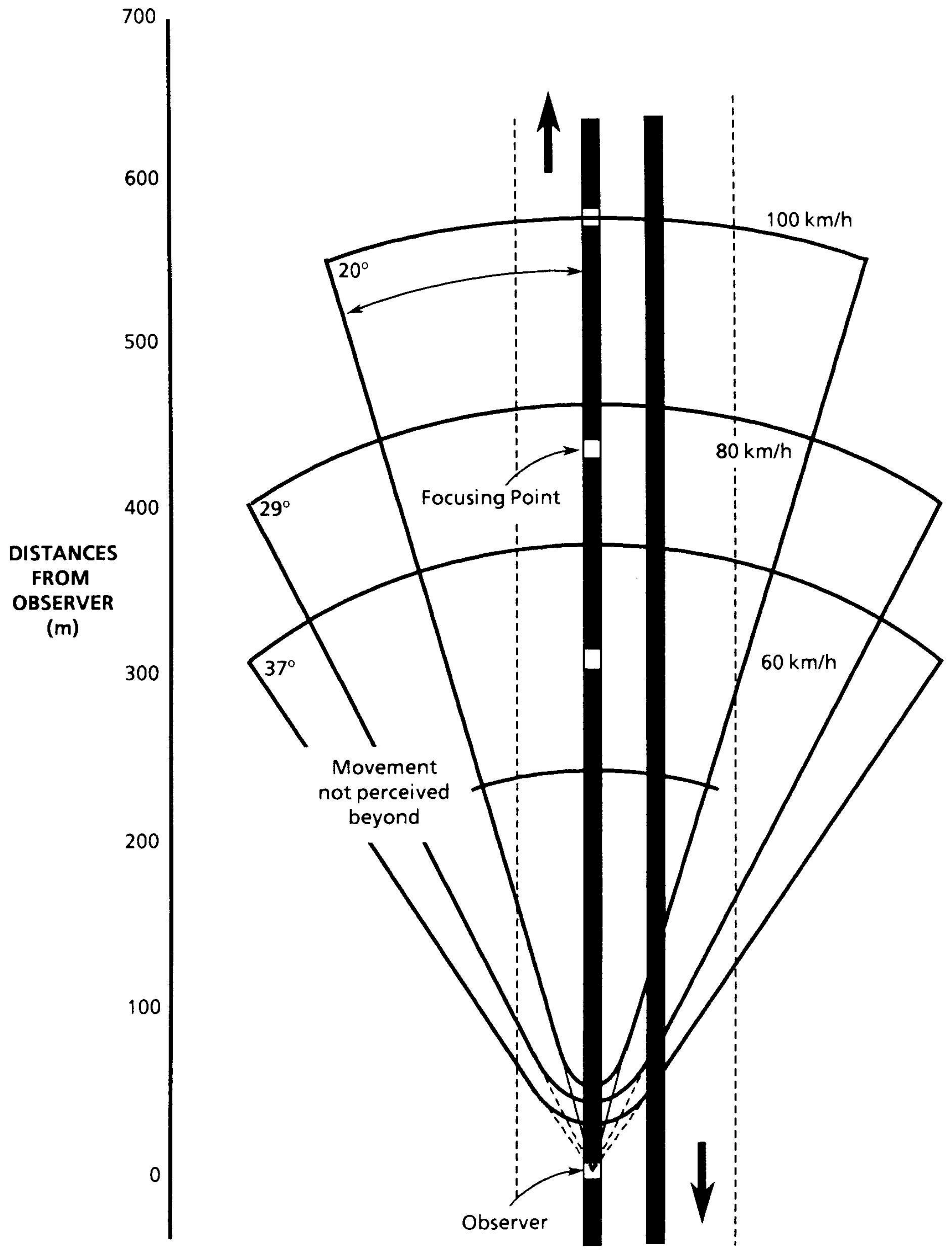
(b) 2.0 seconds: An absolute minimum reaction time which caters generally for older drivers. It may be used where design speeds are less than or equal to 70 km/h, ie. urban areas and low speed rural areas where drivers can be expected to be more alert and better prepared for the unexpected.

2.5.4 Decision Making

Drivers have the ability to select the correct action from a number of alternatives. However, if they are presented with a number of options confusion and error can often result.

A road layout, or signposting arrangement, should therefore:

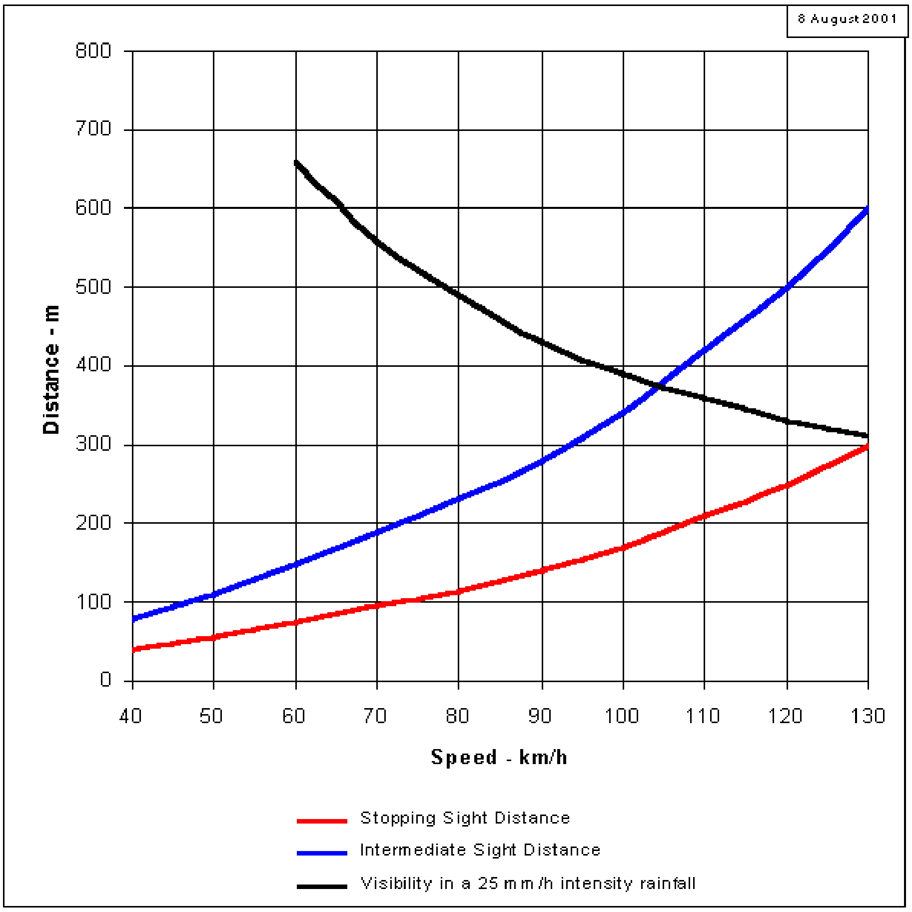
- require a driver to make only one decision at a time, and
- allow at least three seconds travel time between each decision.



Relationship between focusing distance, angle of vision, and distance of foreground detail at speeds of 60 km/h, 80 km/h and 100 km/h.

Source: RTA Road Design Guide

Figure 2.3: Cones of Visions for Various Travel Speeds



Source: RTA Road Design Guide (with NZ Sight Distances)

Figure 2.4: Comparison of Stopping and Intermediate Sight Distances with Visibility at Rainfall Intensity of 25 mm/h

2.6 Speed Parameters

2.6.1 General

To ensure safe road design the various features and elements which control or influence traffic operations must be related to the actual vehicle travel speeds on the road. The speed at which drivers choose to travel a section of road is generally a compromise between:

- the maximum speed at which they are prepared to travel to reach their destination, and
- the perceived level of risk they are prepared to accept.

The level of risk increases as speed increases and in the presence of features that are perceived to contribute to risk, drivers will normally tend to reduce their travel speed. A reduction to the overall geometric standard of a road is seen by drivers as one of the main features contributing to risk and affecting travel speed.

2.6.2 Speed Relationships

As a logical basis for the selection of design speeds four speed parameters are defined. These are:

(a) Desired Speed

Desired speed is the maximum speed drivers will adopt on the less constrained elements of a more or less uniform section of road, ie. the longer straights and large radius horizontal curves, ***when they not constrained by other vehicles***. It is therefore mainly a function of terrain characteristics and the overall geometric standard of the road.

Desired speed is an individual selection but it will lie within a defined speed range on any particular section of road. It is determined by a combination of factors, including:

(i) a driver's assessment of the road's layout and condition,
(ii) the power of their vehicle, and
(iii) the speed at which they will choose to travel the road.

(b) Free Speed

Free speed is the speed adopted by drivers on a particular section of road when constrained only by the alignment. It can be influenced by the driver, the vehicle and the environment, but not by the presence of other traffic. Free speed will therefore be less than or equal to the desired speed for a section of road.

(c) Design Speed (Operating Speed)

Design speed is the speed applied to individual geometric elements which make up a road alignment. It must be a speed that is unlikely to be exceeded by most drivers and should be not less than the estimated 85th percentile of the speed distribution that will result from the construction of a particular geometric element within a given speed environment.

Design speed is used to co-ordinate sight distance, radius, superelevation and friction demand road so that drivers negotiating each element at that speed will not be exposed to unexpected hazards.

The design speed of successive geometric elements should not differ by more than 10 km/h where drivers are expected to vary their speed as they travel along a section of road.

Typical design speeds used for various combinations of road type and terrain are shown in Table 2.5.

Road Type	Environment			
	Rural			Urban
	Terrain			All
	Flat	Rolling	Mountainous	
Two-lane:				
Low Cost *	80	60	40	50
Minor	100	80	60	60
Major	120	110	100	70
Dual Carriageway	120+	110+	100	70 - 80

* *Roads in remote areas and very low volume roads.*

Table 2.5: Typical Design Speeds

(d) Safe Speed

Safe speed is defined as the speed at which a vehicle could travel on a curve of given radius and superelevation e when using the maximum allowable side friction f_{Max} for that design speed. It is equal to the design speed at the minimum radius for the design speed and higher than the design speed at any radii greater than the minimum.

Refer to Section 2.8.4 (d) Design Speed Superelevation for details of the convention adopted by Transit that, at greater than minimum radius, the proportion of sideways force balanced by side friction is the same as at the limiting radius, ie. $\frac{e}{e+f} = \frac{e_{Max}}{e_{Max}+f_{Max}}$.

The ratio $\frac{safe\ speed}{design\ speed}$ is therefore an indication of the margin of safety inherent in the curve design.

2.6.3 Speed Environment

(a) General

The term speed environment is used to describe a characteristic of a section of road. It is regarded as being uniform over a section of road that is reasonably consistent in both terrain and general geometric standard.

Measured free speeds approximately follow a normal distribution. The 85th percentile speed of a normal free speed distribution is very close to the point of inflection of a normal distribution curve and therefore represents the point where increases in speed value cater for a rapidly diminishing proportion of drivers. Speed environment and design speed are both defined by the 85th percentile value of their respective distributions.

Speed environment is numerically equal to the desired speed of the 85th percentile driver over a section of road and is, by definition, equal to the 85th percentile of the observed free speed distribution on the longer straights or large radius curves on that section of road, at low traffic volumes. It is a basic parameter of roading design and can be measured on existing roads but must be estimated for the design of a new road.

NOTE: ***Speed environment Is a description applied to a section of road and not to individual geometric elements.***

Speed environment is influenced by:

(i) the geographic location of a section of road relative to the larger population centres, eg. higher speeds tend to occur more frequently on the more remote sections of roads,
(ii) the general topography of the area, eg. flat open country is conducive to high operating speeds, and
(iii) the general standard of the road, eg. the combination of alignment, cross-section, pavement condition, frequency of intersections and access points.

(b) High Speed Environments

The measured sustained 85th percentile speeds on rural roads rarely exceed 120 km/h. On roads of consistently high geometric standard the speeds on curves and straights are not significantly different. The speed environment and design speeds for the individual geometric elements therefore do not normally need to exceed 120 km/h. However, design speeds up to 130 km/h and higher may be used, these are regarded as notional vehicle travel speeds and roads designed to these standards will provide higher safety factors and levels of service to all drivers.

Normally, design speeds greater than 120 km/h will be only be used for one of two reasons:

(i) where high a geometric standard is compatible with the terrain, and
(ii) where the importance of the road is considered to justify the additional costs of achieving a higher level of service, eg. expressways and motorways.

(c) Intermediate and Low Speed Environments

The speeds adopted by driver's on roads in speed environments of less than 100 km/h will vary in a direct relationship to the characteristics of the roads and their surroundings. In these environments road design should follow the procedure described in Section 4.7.

The 85th percentile speed used as the design speed to co-ordinate the geometric features of a road alignment is mainly a function of two parameters:

(i) speed environment, and
(ii) horizontal curve radius.

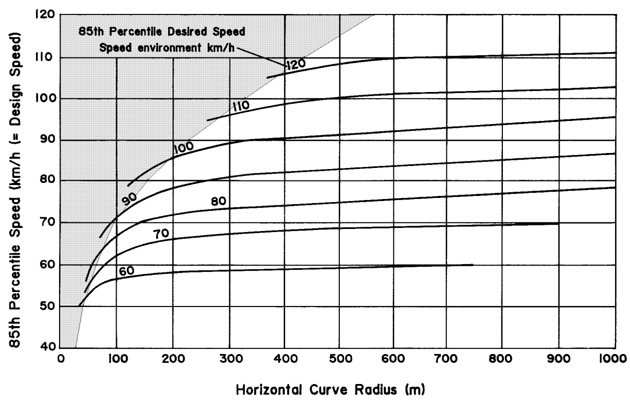
Drivers react to visible cues or hazards they can see on the road ahead and their most important visible cue to selecting an appropriate travel speed is the apparent curvature of the road ahead.

Sight distance has a very limited effect on observed speeds. Drivers are very unlikely to react to a potential hazard which is out of their field of view, eg. a hazard hidden beyond a crest vertical curve, even though their attention may be drawn to it by signs and other such warning devices.

Traffic volumes can have a significant effect on observed speeds but they are not normally considered during geometric road design because design speeds must be related to the free speeds of vehicles at low traffic flows and when they are not constrained by other vehicles. Traffic volumes are however considered during capacity analysis, to determine the number of traffic lanes needed for the road.

Design speeds in intermediate and low speed environments will vary in accordance with the predicted 85th percentile speed of the individual geometric elements. To ensure safe operating conditions in these situations the incremental changes in design speed for successive geometric elements must be limited. Methods to achieve this are described in Section 4.7.

The relationship between speed environment, horizontal curve radius and design speed is illustrated in Figure 2.4.



Source: Austroads Rural Road Design Guide

Figure 2.5: Speed Environment / Horizontal Curve Radius / Design Speed Relationship

2.6.4 Estimating Vehicle Speeds

Travel times and/or speeds should be measured wherever possible. Suitable methods for measuring average travel times and speeds include:

- **(i)** floating car surveys,
- **(ii)** number plate surveys, and
- **(iii)** spot measurement of speeds.

Floating car and number plate surveys can be used to measure average travel time over a length of road.

A floating car survey is a relatively cheap and convenient method. It will not, however, readily differentiate the average travel times of light and heavy vehicles and is really only suitable for use with higher traffic volumes, ie. those greater than 500 vehicles/hour/lane.

A number plate survey is much a larger undertaking but it is potentially more accurate and has the ability to give average travel times for individual or categories of vehicle. The average travel time over a section of road may not provide sufficient information for calculating vehicle operating costs if one or more speed change cycles occur within the section. In these cases spot speed measurements will be required at a several locations to establish the average cruise speed, and the points of minimum speed, for the road section.

An alternative to spot speed measurements is to arrange number plate survey points so as to avoid speed change cycles within those lengths of road.

2.7 Design Speed

2.7.1 General

Design speed is defined as the speed selected to establish specific minimum geometric design elements for a particular section of road. These design elements include the vertical and horizontal alignments and sight distance. Other features such as widths of pavement and shoulders, horizontal clearances, AADT, etc., are generally not directly related to design speed.

The choice of design speed is influenced mainly by the type of terrain, economic considerations, environmental factors, type and anticipated volume of traffic, functional classification of the road, and whether the road is in an rural or urban area. A road in level or rolling terrain justifies a higher design speed than one in mountainous terrain. Where a difficult location is obvious to approaching drivers, they are more apt to accept a lower design speed than where there is no apparent reason for it. Scenic values are also a consideration in the selection of a design speed.

A road carrying a large volume of traffic may justify a higher design speed than a less important facility in similar topography, particularly where the savings in vehicle operation and other costs are sufficient to offset the increased cost of land purchase and construction. However, a lower design speed must never be used for any road where the topography is such that drivers are likely to travel at high speeds.

Subject to the above considerations, as high a design speed as possible should be used. It is preferable that the design speed for any section of road be a constant value and a constant design speed must be used for dual carriageway roads and sections of high speed road. Design speeds may vary on sections of intermediate and low speed roads, as described in Sections 2.7.3 and 2.7.4.

During the detailed design phase of a project special situations may arise in which engineering, economic, environmental, or other considerations make it impractical to meet the minimum standards required for the design speed. The most likely examples of this type of situation are partial or brief horizontal sight distance restrictions, such as those imposed by bridge rails, bridge columns, retaining walls, sound walls, cut slopes, and median barriers. The cost to correct such restrictions may not be justified and technically this will result in a reduction in the effective design speed at the location in question. These situations must be documented and approved by the Strategy and Standards Division Traffic and Design Manager before reduced design standards can be implemented.

Research at the Australian Road Research Board (ARRB) has shown that there is a maximum speed at which drivers are likely to travel on a road of relatively uniform geometric standard and terrain characteristics, even when free of other traffic and alignment constraints. This speed is the desired speed of drivers on that particular road section and it is likely to be achieved only on the reasonably long straight sections. It is unlikely to exceed 120 km/h on any type of road.

The desired speed is determined by the drivers perception of the overall standard of the section of road and can be related to the standard of the geometric elements (mainly the horizontal curves) and the road environment (the terrain).

The 85th percentile desired speed of drivers on a section of road with a relatively uniform terrain features and geometric standard is defined as being numerically equal to the Speed Environment for that section of road.

Research findings and accumulated design experience suggests that there are effectively three classes of road and that different design philosophies should be employed for each. All have the fundamental objective of providing a road which conforms to driver expectancies.

2.7.2 High Speed Roads

On high standard roads, ie. where design speeds are 100 km/h or higher, drivers tend to adopt a relatively uniform travel speed. This will generally be less than the speed assumed for the design of individual geometric elements but drivers will expect to be able to maintain a high travel speed on this type of road. The road's design must therefore ensure that this expectation can be safely met along the entire length of the road.

An increase in design standard is not likely to produce a commensurate increase in travel speeds but it will provide a higher level of safety and convenience to all road users.

2.7.3 Intermediate Speed Roads

On roads where travel speeds are 100 km/h or less the effects of terrain, general geometric standard and the radius of individual horizontal curves tend to cause drivers to vary their speed as they travel along the road. This variation in travel speed should be considered during the design of individual geometric roading elements and, provided standards in keeping with driver expectancies are adopted, a safe and adequate alignment will generally result.

A design method has been developed for rural roads which uses the speed environment of the section of road to predict an 85th percentile travel speed for each alignment element and that speed is then used as the design speed for the element. This method ensures compatibility between the standards adopted for the design of individual elements and observed driver speed behaviour, see Section 4.7 for details.

2.7.4 Low Speed Roads

Low speed roads, ie. those where travel speeds are less than about 70 km/h, are normally only found in difficult terrain conditions where costs preclude the adoption of anything better. The road alignment in these circumstances can be expected to produce a high degree of driver alertness and the low standards are therefore both expected and acceptable.

The most pragmatic approach to road design in these constrained situations is to:

- **(i)** provide the best standard that appears to be practicable for each individual geometric element in an initial scheme design,
- **(ii)** check that these meet the minimum standards for the predicted speed environment, and
- **(iii)** modify as necessary to ensure absolute minimum standards are met.

Innovative, non-standard treatments will often be required to when minimum standards cannot met in the scheme design.

2.7.5 Design Approach

The speed environment of a section of road determines the design standard for each geometric element within that section of road. This provides a rational and consistent basis for low and intermediate-speed roads and is the approach used in this manual.

The speed environment approach can also be logically extended to high-speed roads where a constant design speed is often used. In these cases the speed environment forms the lower bound of speed assumed for design purposes. The difference between this and the actual design speed represents the additional safety and convenience or quality of service that is provided on these roads.

Good road design, especially under constraints, involves judgement and compromise between conflicting goals. Experience assists a designer to arrive at an appropriate compromise that cannot be met by a system of mathematical rules. Therefore, this manual gives ranges of values within which the designer has reasonable flexibility to produce an appropriate design for a specific problem while still retaining an acceptable overall level of uniformity.

2.8 Horizontal Alignment

2.8.1 General

The horizontal alignment of a road is usually a series of straights (tangents) and circular curves connected by transition curves (spirals). An alignment without straight sections is described as curvilinear and it is most suited to divided roads and two lane roads in flat country. It may also be a suitable alignment for two lane roads in undulating country when overtaking provisions are not impaired because it can help to reduce headlight glare and aid recognition of the speed of approach of opposing vehicles in some cases.

2.8.2 Horizontal Curvature

The forces acting on a vehicle while negotiating a circular path are shown in Figure 2.6.

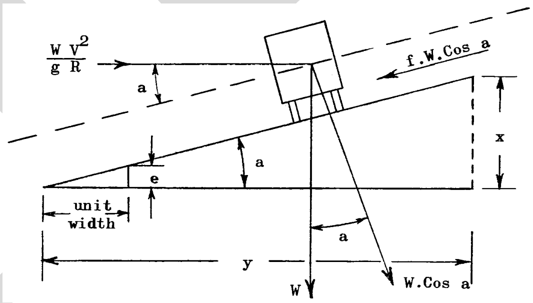


Figure 2.6: Forces Acting on a Vehicle on a Circular Curve

A sideways force tends to throw the vehicle outwards from the centre of the curve. This force (P) is dependent on the speed of the vehicle (V), the radius of its path (R), and the mass of the vehicle which may be expressed as its weight (W) divided by the acceleration due to gravity (g) according to the following formula: $P = \frac{W V^2}{g R}$

On a superelevated section of road this sideways force is balanced partly by the crossfall and partly by the friction developed between the tyres and the pavement. For stability, ie. for a vehicle to safely negotiate a circular arc of radius R, the sum of the crossfall rate (e) and limiting side friction (f) must exceed: $\frac{V^2}{g R}$

Resolving forces parallel to the road surface gives the following equation:

$$\frac{WV^2}{gR} \times Cos\ a = (W \times Sin\ a) + (f \times W \times Cos\ a)$$

$$\therefore \quad \frac{WV^2}{gR} = (W \times \frac{Sin\ a}{Cos\ a}) + (f \times W \times Cos\ a)$$

$$= (W \times Tan\ a) + (f \times W)$$

or $$\frac{V^2}{gR} = (Tan\ a + f)$$

But $$Tan\ a = \frac{x}{y} = \frac{e}{1} = e$$

$$\therefore \text{ Centrifugal Ratio } = \frac{P}{W} = \frac{V^2}{gR} = e + f$$

and by rearranging: $$R = \frac{V^2}{g(e + f)}$$

Substituting for g and rounding off gives:

$$R = \frac{V^2}{g\ (e + f)} \quad \ldots\ldots\ldots\ldots \quad \textbf{(1)}$$

Where:
R = m
V = km/h
e = m/m
f = unbalanced coefficient of side friction

Maximum superelevation e_{Max} is set as 0.10 by standard because higher values may cause large slow moving or stopped heavy vehicles, eg. stock transporters, to overturn. If the limiting values of e and f are substituted into Equation ***(1)*** the minimum radius allowable for any speed value can be calculated.

eg. at 80 km/h f = 0.26 and e = 0.10

$$\therefore \quad R_{(80\ min)} = \frac{80^2}{127\ (0.10 + 0.26)} = 140\ m$$

NOTE: ***e and f are set by standard and*** R_{min} ***is a result.***

2.8.3 Side Friction

The circular path formula assumes that the path of a vehicle traversing a curve assumes a constant radius equal to the curve radius. However, this does not occur in practice.

On short, small-radius curves, drivers tend to use the available lane width so that vehicle path radius is increased and the actual f is less than the value implied by the circular path formula. For longer, large-radius curves, however, drivers must make additional steering adjustments to maintain the position of the vehicle within the lane. These tend to decrease the radius of the vehicle path so that the actual f is greater than the value implied by the formula.

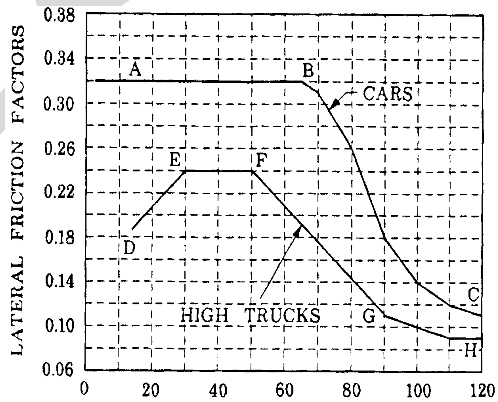
The differences between the conditions assumed for the circular path formula and what actually occurs in practice mean that f values appropriate for use in the formula cannot be derived directly from known pavement skid resistance.

However, drivers learn to assess the speed appropriate to a given curve, and this forms the basis of the design criteria. From measurements of actual vehicle speed and curve geometry, the circular path formula can be used to compute the equivalent f value which driver's regard as appropriate.

There are practical limits to the speeds at which vehicles can safely travel around curves on normal roads. Tests have shown that most cars will develop instability when f exceeds 0.6 while vehicles with a high centre of gravity can develop roll instability when f is as low as 0.2.

Figure 2.7 illustrates the different f requirements of trucks and cars and how they vary with speed. The car and small truck factors between points A and B are based on speeds measured by *Vicoads* at roundabouts while the factors between points B and C are based on measurements made by researchers at *ARRB*. The truck factors are those used by *Vicroads* and are based on the following assumptions:

- Point D is determined by a speed of 15 km/h on a 15 m radius curve with an assumed superelevation rate of -5%,
- Points E and F are based on the rollover threshold for a large truck with homogeneous load, and
- Points G and H fixed to provide radii consistent with car requirements at 100 km/h, taking into account the speed differential between cars and trucks.



Source: Vicroads

Figure 2.7: Variation of Side Friction Requirement with Speed

The *'Passenger Car'* f values shown in Table 2.6 are used for state highway road design work. These sealed road values have been derived from observed driver speed behaviour by the ARRB and are appropriate for use with the circular path formula. The values for speeds less than 90 km/h are those typically used by vehicles travelling at the 85th percentile speed on the sharper curves in a particular speed environment. The values shown for higher speeds are in excess of those likely to be required by the 85th percentile driver and are in keeping with the concept that high speed alignments should provide a high degree of comfort and safety for all road users. The *'Unsealed Road'* and *'Truck'* values shown in Table 2.6 are those used by Vicroads and cannot be verified. They are, however, suggested as best practice values for use an appropriate situation, in the absence of anything better.

Design Speed (km/h)	f		
	Passenger Cars		Trucks
	Sealed Roads	Unsealed Roads	
15	0.35	0.09	0.16
20	0.35	0.10	0.19
25	0.35	0.11	0.21
30	0.35	0.12	0.24
35	0.35	0.12	0.24
40	0.35	0.12	0.24
50	0.35	0.12	0.24
60	0.33	0.11	0.24
70	0.31	0.10	0.18
80	0.26	0.10	0.14
90	0.18	0.09	0.11
100	0.14	0.09	0.10
110	0.12	0.08	0.09
120	0.11	0.08	0.09
130	0.11	0.07	0.08

Source: VicRoads

Table 2.6: Maximum Design Speed Values of Side Friction

NOTE: The f values shown in Table 2.6:

- ***are maximum values and should only used in the limiting case, ie. the minimum radius curve for a particular design speed in conjunction with the maximum allowable rate of superelevation (e), and***
- ***assume that the construction and maintenance techniques used on sealed roads will ensure an adequate factor of safety against skidding. The susceptibility of the wearing surface to polishing, the macro-texture of the surface and the amount of bitumen used in the wearing surface are all important matters in the initial construction of a pavement which contribute to skid resistance. Freedom from contamination by oil spillage or loose aggregate, and resealing when surface texture becomes too smooth, are important aspects in maintenance of skid resistance. Normally, a pavement which is properly maintained will retain adequate resistance to skidding under all but extreme conditions of driver behaviour or weather.***

2.8.4 Superelevation

(a) General

Superelevation is applied primarily for safety, but other factors are comfort and appearance. The superelevation applied to a road needs to take into account:

- the design speed of the curve, which is taken as the speed at which the 85th percentile driver is expected to negotiate it,
- the tendency of very slow moving vehicles to track towards the centre,
- the stability of high laden commercial vehicles, particularly those that could have a movable load, eg. stock transporters,
- differences between inner and outer formation levels, especially in flat country, and
- the length available to introduce the necessary superelevation.

(b) Maximum Superelevation

The use of maximum rate superelevation is only necessary when the curve radius is approaching the minimum for the design speed, ie. towards the left hand end of the speed environment/design speed curve segments of Figure 2.5. Normally, this will occur only in steep terrain or where there are constraints on increasing the radius of an individual curve in a group of curves. In such cases, if superelevation approaching the maximum is not applied, there may be a sharp increase in side friction demand by a vehicle travelling at the design speed as it enters the curve in question. While it can be shown that the side friction value may be within the normal range drivers use, a sudden change in side friction demand will lead to a sudden change in steering attitude of a vehicle negotiating the road. Large fluctuations in side friction demand are best avoided, as a driver may not anticipate a sudden change in the response of his vehicle.

The general maximum superelevation rates for all new state highway work are:

(i) Two-lane two-way roads: 10%, with 6% to 7% being the preferred maximum in flat terrain.

(ii) Dual carriageway roads: 8%.

NOTE:

For rehabilitation work on existing roads, eg. shape correction, pavement strengthening, etc, and in mountainous terrain, the use of superelevation rates up to 12% may sometimes be necessary. In these situations designers must ensure that the use of high superelevation rates in conjunction with steep grades do not lead to large vehicle stability problems.

(c) Minimum Superelevation

It is desirable to superelevate all curves to at least the value of the normal pavement crossfall rate used on straights, usually 3%. The use of normal crossfall, ie. -3% superelevation, may only be considered when curve radii exceed the values given in Table 2.7.

Design Speed (km/h)	30	40	50	60	70	80	90	100	110	120	130
Minimum Radius (m)	200	350	550	800	1100	1500	1900	2400	3000	3700	4500

Table 2.7: Minimum Radius for -3% Superelevation

(NOTE: Min. radii calculated using e = -0.03 and (e + f) = 0.6 to 0.7 (Austroads recommendtion)

(d) Design Speed Superelevation

Any radius greater than $R_{(Min)}$ *is* acceptable for the design of a curve at a given speed value. By rearranging Equation (***1***) it can be seen that for R greater than $R_{(Min)}$ the sum of side friction and superelevation will be less than the limiting values applied at $R_{(Min)}$:

$$(e + f) = \frac{V^2}{127R}$$

In common with many other roading authorities Transit has adopted the convention that at greater than minimum radius the proportion of sideways force balanced by side friction is the same as at the limiting radius. In other words $\frac{e}{e + f}$ is constant.

Superelevation is usually expressed as a percentage and, for all design speeds, is calculated by the following formula:

$$e_{(\%)} = \frac{V^2 \times S_k}{1.27\ R}$$

Where: V = design speed (km/h)
R = curve radius (m)
S_k = the ratio of maximum superelevation to the centrifugal ratio.

$$= \frac{e_{Max}}{e_{Max} + f_{Max}}$$

Standard S_k values are listed in Table 2.8.

V (km/h)	30, 40 & 50	60	70	80	90	100	110	120 & 130
S_k	0.222	0.233	0.244	0.278	0.357	0.417	0.455	0.476

Table 2.8: S_k Values for the Calculation of Design Speed Superelevation

(e) Application of Superelevation

Superelevation must be applied safely and in a manner that does not cause discomfort to vehicle occupants, or produce a visually unsightly road alignment.

The rate of application of superelevation on two-lane two-way roads should not normally exceed 2.5% per second of travel at design speed. Rates up to a maximum of 3.5% per second may, however, be used in constrained conditions where the design speed is ≤ 70 km/h, and in some other specially approved situations. Superelevation is applied over the length calculated by the method described in Section 4.6.3 (d) (v).

Divided roads usually have wider pavements than two-lane two-way roads and are designed to higher standards. On these roads an enhanced vehicle occupant comfort is used, ie. a pavement rotation rate of not more than 2.0% per second of travel at design speed, and the visual appearance of the road, ie. the relative grade between the inner and outer edges of the travelled way, is also considered. The length of road needed satisfy the relative grade criteria is calculated by the method described in Section 4.6.3 (d) (vi) and superelevation is applied over the longer of the lengths determined by the above criteria.

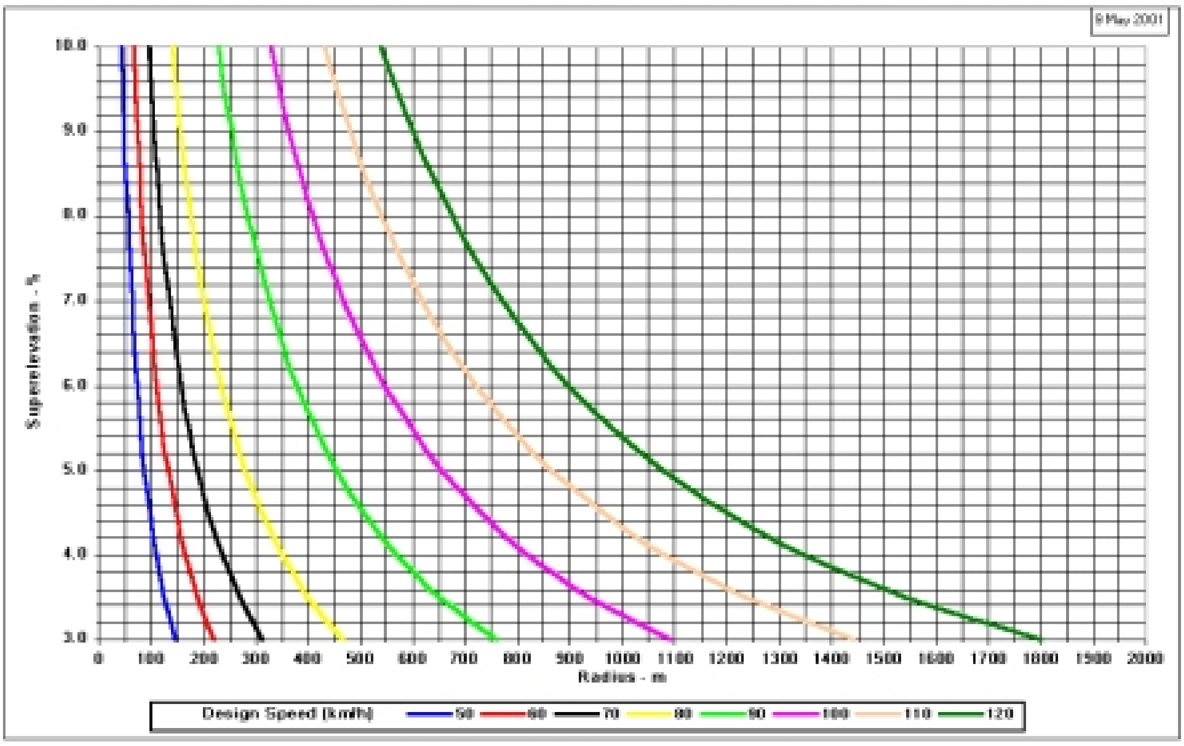


Figure 2.8: Design Speed / Curve Radius / Superelevation Relationship for Two-lane Two-way Roads

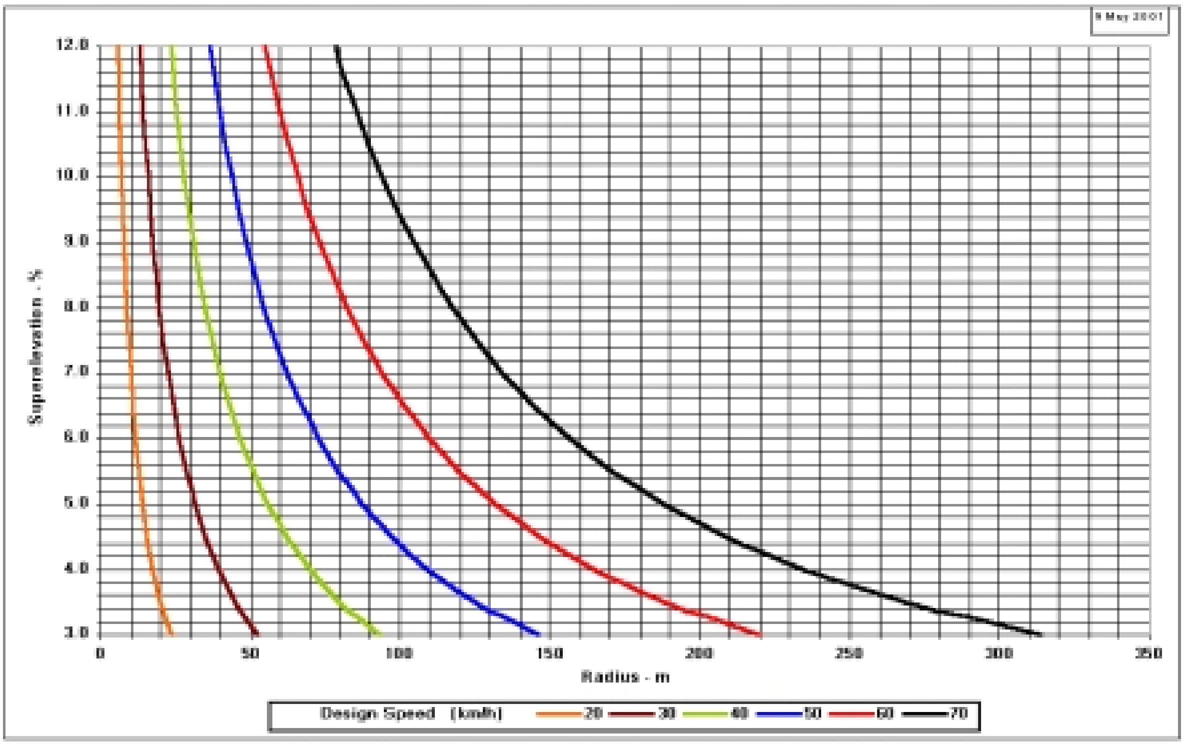


Figure 2.9: Design Speed / Curve Radius / Superelevation Relationship for Two-lane Two-way Roads in Mountainous and other Low Speed Situations where Design Speed is ≤ 70 km/h

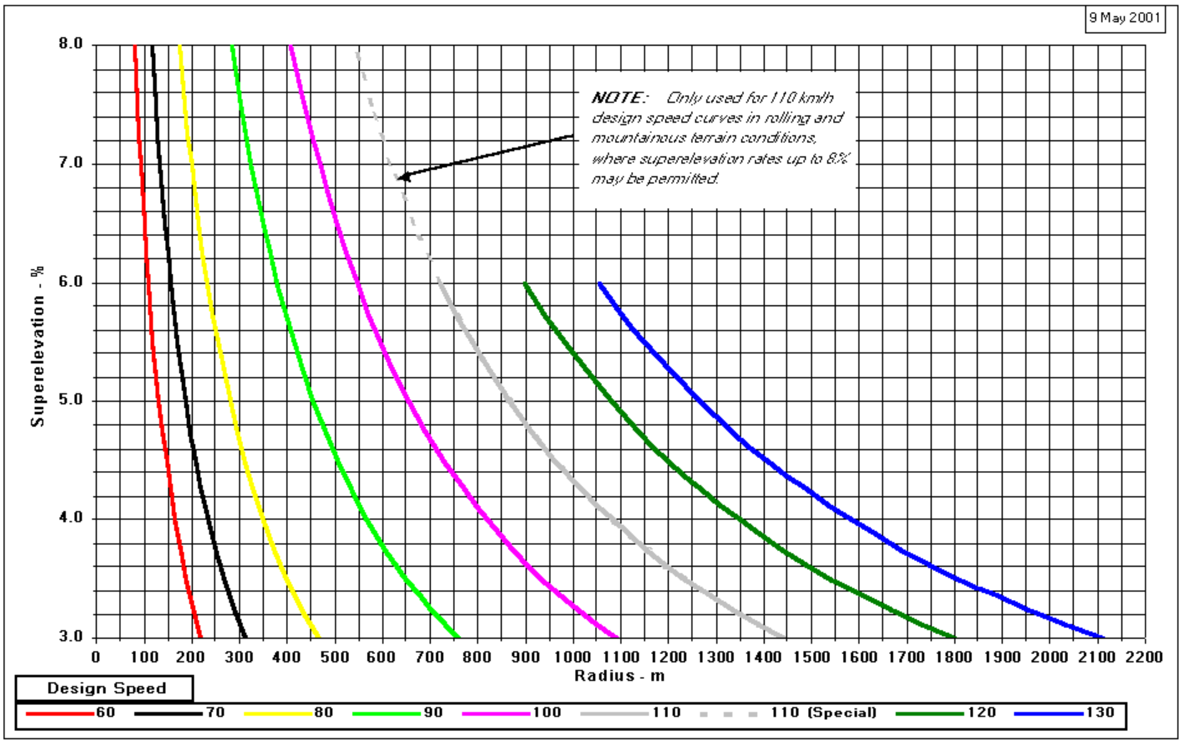


Figure 2.10: Design Speed / Curve Radius / Superelevation Relationship for Divided Roads

2.8.5 Transition Curves

(a) General

The plan transition is the length of road over which the curve radius is gradually changed from infinity on the tangent to that of the circular arc. By convention, but not necessarily, it is also the length over which the superelevation is applied

A cubic spiral transition is normally used for a transition, for no better reason than its use in many computer programs. In practical terms there is no real significant difference between the spiral and other curves that are also sometimes used, eg. the lemniscate.

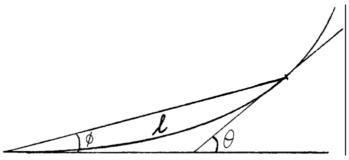


Figure 2.11: Basic Spiral Curve Detail

The basic equations of the spiral is:

$$\theta = \frac{l^2}{2(R \times SL)} \quad \text{and} \quad r\,l = \text{a constant}$$

Where:
- R is the minimum radius (circular arc)
- SL is the length to minimum radius
- l is any length of spiral
- θ is the deflection angle
- r is radius at a distance l

There is also an approximation which holds true for transition type curves and this is:

$$l = 6R\alpha \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (2)$$

where α is a small angle expressed in radians.

(b) Unit Chord

A unit chord is defined as the length of the polar ray given by a deflection angle of 16´ of arc. At this very small angle the polar ray approximates closely to the length of the spiral and substituting in Equation *(2)* gives:

$$r = \frac{l}{6 \times \frac{\pi}{180} \times \frac{16}{60}}$$

$$= 35.81$$

ie. at a length of 1 unit chord
the radius is 35.81 unit chords.

Hence, RL = 35.81 when R and SL are measured in unit chords and when R and L are measured in metres:

$$\frac{(R \times SL)}{\text{unit chord}^2} = 35.81$$

or

$$\text{unit chord} = \sqrt{\frac{(R \times SL)}{35.81}} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (3)$$

This constant, and the normal mathematics of the spiral, are used to calculate the other elements of the spiral transition in terms of unit chords. These are listed in Table 4.4: *Dimensions of Unit Chord Spiral and Central Circular Arc.*

Ignoring the mathematics, it is important to understand what has been done. By adopting the unit chord as the unit of measurement all possible spiral transitions have been reduced to a single spiral. The value chosen for the unit chord scales this single spiral to a size appropriate for the design speed. A greater or lesser length of the scaled spiral is used, depending on the radius it must match with.

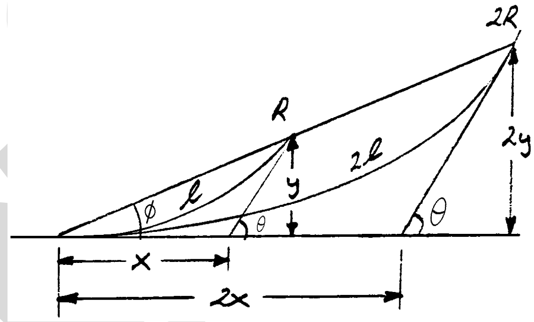


Figure 2.12: Unit Chord Spiral Curve Detail

The approach adopted by many roading authorities, including Transit New Zealand, is to set the minimum unit chord for a given design speed to a value which will ensure that superelevation is applied safely and in a manner that will not cause discomfort to vehicle occupants or produce a visually unsightly road alignment. Refer to Section 2.8.4 (e) above for details of how superelevation is applied on various types of road.

The minimum radius for any design speed, $R_{(Min)}$, is achieved with the maximum allowable superelevation of 10%. Conventional design practice requires zero crossfall at the tangent point of a transition curve and maximum superelevation at the point where the transition curve joins the circular arc. The time taken to rotate the pavement between these points at the specified rate, in seconds, multiplied by the design speed, in m/sec, gives the minimum transition length $L_{(Min)}$. Substituting $R_{(Min)}$ and $L_{(Min)}$ in equation **(3)** gives the minimum unit chord for the design speed.

eg. The minimum curve radius for a two-lane two-way road at a design speed of 80 km/h is 140 m. The time to rotate the road pavement from 0% to 10% superelevation at 2.5% per second at this speed is:

$$\frac{10}{2.5\%\ \text{per sec.}} = 4\ \text{seconds}$$

The distance travelled in this time is the minimum spiral transition length, and is:

$$4 \times \frac{80}{3.6} = 88.89\ m$$

The minimum unit chord for an 80 km/h design speed horizontal curve is therefore:

$$\sqrt{\frac{(140 \times 88.89)}{35.81}} = 18.6\ m$$

The use of the appropriate unit chord will ensure the correct matching of transition spiral length, circular arc radius, superelevation rate and side friction demand at any design speed. Unit chords also provide a quick and easy means of checking horizontal curve designs.

Design standards for horizontal curves on state highways are given in Tables 2.9 and 2.10 below.

Design Speed (km/h)	Maximum Superelevation (%)	Maximum Coefficient of Side Friction	Minimum Radius (m)	Minimum Unit Chord for a Maximum Rate of Rotation of Road Crossfall of:	
				2.5% / second *(Normal Design Conditions)*	**3.5% / second** *(Constrained Design Conditions Only)*
30	10 [1]	0.35	16	4.5	3.2
40	10 [1]	0.35	28	6.3	5.0
50	10 [1]	0.35	44	8.2	7.0
60	10 [1]	0.33	66	11.1	.2
70	10	0.31	95	14.3	11.8
80	10	0.26	140	18.6	***14.8*** [2]
90	10	0.18	228	25.2	N/A
100	10	0.14	328	31.9	N/A
110	10	0.12	433	38.4	N/A
120	10	0.11	540	44.8	N/A
130	10	0.11	634	50.6	N/A

[1] Up to 12% may be approved for use in some situations. *[2] Requires specific approval in ALL cases.*

Table 2.9: Horizontal Curve Design Standards for Two-lane Two-way State Highways, including Climbing Lanes but not Passing Lanes

Design Speed (km/h)	Maximum Superelevation (%)	Coefficient of Side Friction required at Minimum Radius	Minimum Radius (m)	Minimum Unit Chord (m)	
				Four-lane roads	Six -lane roads
60	8	*0.26*	82	12.4	14.0
70	8	*0.25*	118	16.0	17.5
80	8	*0.21*	175	20.8	22.2
90	8	*0.14*	285	28.2	29.5
100	8	*0.11*	410	35.7	36.6
110	Rolling / Mountainous terrain - 8 Flat terrain - 6	*0.10* *0.07*	541 722	40..3	43.6
120	6	*0.07*	900	50.1	
130	6	*0.07*	1056	56.5	

Table 2.10: Horizontal Curve Design Standards for Divided Road State Highways, and Passing Lanes on Two-lane Two-way State Highways

NOTES:

1. ***In constrained design conditions, ie. mountainous terrain and other similar low speed situations, pavement rotation rates up to 3.5% per second may be used with design speeds $\leq$ 70 km/h. Pavement rotation rates up to 3.5% per second with a design speed of 80 km/h require written justification explaining why the standard design criteria need to be varied and prior approval by the Highway Policy Division Traffic and Design Manager before being used.***
2. ***Curve design standards for divided road are based on a pavement rotation rate of 2% per second or the relative grade between inner and outer edges of the traffic lanes way on the one-way pavement, whichever gives the largest unit chord.***

2.9 Sight Distance

2.9.1 General

In practice, sight distance has little effect on the actual speeds adopted by drivers on the road, speed is influenced mainly by the geometric features of the road layout

Ensuring drivers are able to perceive a hazard on the road ahead in sufficient time to safely avoid a mishap is a however very important aspect of road design. For road design purposes sight distance is defined as the distance at which a driver will be able to perceive an object of specified height on the road ahead and then safely stop their vehicle before it reaches the object, assuming adequate light and visual acuity and clear atmospheric conditions.

A driver's view of the road and its surroundings is dependent on both vehicle and road design. Dangerous blind spots in the driver's field of vision can be, and are, caused by the placement and shape of vehicle roof supports and the height and shape of the backs of the front seats.

An object within a driver's field view may not always be perceived. It has been shown that when a driver is travelling on sharp curves or in when their vehicle is rapidly accelerating or decelerating, ie. when they are subject to unusual forces, their ability to perceive an object can be reduced because the mechanism of sight is upset. Fatigue and drugs also add to the time of perception and therefore may increase a drivers total reaction time.

Peripheral vision declines as speed increases and drivers have some difficulty in perceiving small hazards. At intersections and level crossings a driver may not perceive a movement on the side, even though it is within their field of view.

As a general rule sight distance should be made as long as practicable and using a design speed 10 to 15 km/h greater than the horizontal alignment will achieve this. Sight distance is, however, often restricted by crest vertical curves, horizontal curves in cuttings, roadside vegetation and buildings at intersections which can make manoeuvres, such as overtaking, unsafe. Four sight distance conditions need to be considered in road design, ie. stopping, overtaking, intermediate and headlight. These, and their various parameters, are described in the following sections.

2.9.2 Sight Distance Parameters

(a) General

The basic parameters used for the calculation of sight distance are *'Reaction Time'* and *'Longitudinal Deceleration'*.

(i) Reaction time

Driver reaction time and its derivation are described in more detail in Section 2.5.3: Reaction Time. Two values may be used in state highway road design:

1. A general reaction time of 2.5 seconds which should normally be used in all areas, and must be used in rural areas.

2. An absolute minimum reaction time of 2.0 seconds. This caters generally for older drivers and, with Strategy and Standards manager approval, it may be used where design speeds are less than or equal to 70 km/h, ie. urban areas and low speed rural areas where drivers can be expected to be more alert and better prepared for the unexpected.

(ii) Longitudinal Deceleration

Tests have shown that on good, dry pavements modern passenger cars can consistently achieve deceleration rates in excess of 1.0 g. However, the values used for design purposes need to allow for the degradation of pavement skid resistance when wet and for a reasonable amount of surface polishing.

The coefficient of longitudinal deceleration values which must be used for state highway road design work are given in Table 2.11. These values are less than those given in the Austroads *Guide to the Geometric Design of Rural Roads* because SCRIM testing has shown these values cannot be achieved on New Zealand state highways. The lower values assumed for the higher speeds reflect the reduction in wet pavement skid resistance with increasing speed and the need for drivers to retain lateral vehicle control over the longer braking distances.

Design Speed (km/h)	Coefficient of Longitudinal Deceleration
30	0.52
40	0.52
50	0.52
60	0.48
70	0.45
80	0.43
90	0.41
100	0.39
110	0.37
120	0.35
≥ 130	0.23

Source: Austroads

Table 2.11: Coefficient of Longitudinal Deceleration

(b) Vertical Curves

Two additional sight distance parameters, *'Driver Eye Height'* and *'Object Height'* are used in vertical curve design:

(i) Driver Eye Height

The seating position in a vehicle determines the driver's eye height, ie. the distance between the driver's eye and the road surface. Eye height is therefore as varied as the numerous makes and models of vehicles.

Two representative driver eye heights, based on recent research and consideration of the vehicle fleet, are to be used in state highway road design:

1. 1.05 m for cars, light trucks and van drivers, and
2. 1.8 m for truck and heavy vehicle drivers.

A 1.05 m eye height should be used for general design work. A change of a few millimetres in this value has little influence on sight distance but a similar change in the height assumed for an object on the road does make a significant difference.

(ii) Object Height

The object heights used in the calculation of sight distance are a compromise and five values have been defined to cover the various state highway road design situations:

1. **Intersections:** 0 mm (zero) , ie. markings on the road surface.
2. **General Object Height:** 200 mm.
3. **Oncoming Vehicle:** 1.15 m.
4. **Vehicle Headlight:** 750 mm above the road surface.
5. **Vehicle Tail Lights:** 600 mm above the road surface.

Drivers are only likely to stop if there is a large stationary object, or vehicle, on the roadway. Most will attempt to evade a small object rather than stop.

Sight distances calculated with a zero object height allow for very small hazards, such as surface defects, to be perceived. However, at high speeds these may not be clearly visible to most drivers and many could find it difficult to stop or avoid, them by the time they recognise there is a hazard on the road.

The length of crest vertical curves increases significantly as the object height approaches zero. A 200 mm object height has proved to be a good general dimension that produces satisfactory road designs. However, a zero object height can be used where it is desirable for drivers to be able to see the road markings in advance of intersection traffic islands, and at approaches to causeways and floodways where sand and other debris left by flood water, or even washouts, may occur.

On one way roads, ie. motorways and median divided dual carriageway roads, a driver should be able to see a vehicle stopped on the road ahead in sufficient time to stop before colliding with it. The object height in these situations should range from 1.15 m, ie. vehicle height, to 600 mm, ie. vehicle tail light height.

The only case where the upper limits of driver eye height and object height are relevant is the combination of a sag vertical curve and an overhead obstruction such as a bridge. A driver eye height of 1.8 m and an object height of 600 mm are used in these situations, see Figure 5.11.

2.9.3 Stopping Sight Distance (SSD)

'Stopping Sight Distance' is considered to be the minimum sight distance a driver should have at any point on the road. It is the minimum distance required by an average driver of a vehicle travelling at a given speed to perceive an object on the road ahead, react and stop before reaching it.

Stopping sight distance is measured along the line of travel from a point 1.05 m above the road surface, ie. the drivers eye height, to an object 200 mm high, ie. a stationary object on the road surface.

Stopping distances are affected by road gradient, a downgrade increases braking distance and an upgrade reduces it. A grade adjustment factor is incorporated into the equation used to calculate stopping sight distance.

The sight distance parameters described in Section 2.9.2 are general values and will therefore not be absolutely correct for all situations. They have however been found to work well in practice and have been used to generate the minimum stopping distances on a level grade shown in Table 2.12.

Stopping sight distance has two components, the distance travelled during the driver reaction time and the distance travelled during braking, and is calculated with the following equation:

$$SSD = \frac{R_T\ V}{3.6} + \frac{V^2}{254\ (d \pm 0.01G)}$$

Where:

SSD	=	stopping sight distance (m)
d	=	coefficient of longitudinal deceleration, from Table 2.11
R_T	=	driver reaction time, from Section 2.5.3 (sec).
V	=	initial or design speed of vehicle (km/h)
G	=	longitudinal grade, positive if uphill, negative if downhill (%)

Table 2.12 shows the stopping sight distances required on a ***LEVEL (0%) GRADE***. Adjustments should normally be made for gradient and must be made for sustained downhill grades, ie. > 2 km, steeper than - 3%.

2.9.4 Overtaking Sight Distance (OSD)

At reasonable intervals on a two lane two-way road drivers should have sufficient visibility to the allow the safe and uninterrupted overtaking of another vehicle, ie. they should have *'Overtaking Sight Distance'*.

Human beings cannot readily, or accurately, assess the speed of an oncoming vehicle if it is some distance away and the distance a driver needs to overtake safely is a vexed question.

There are many variables involved in the overtaking distance calculations, including:

- the judgement of the overtaking driver and the risks they is prepared to take,
- the speed and size of vehicles to be overtaken,
- the speed of the overtaking vehicle and the perceived risk during the manoeuvre,
- the speed of a potential oncoming vehicle, and
- the evasive action or braking undertaken by that vehicle and by the overtaken vehicle.

The optimum condition is one in which a driver can follow the vehicle ahead for a short time while assessing the chances of overtaking, pull out, overtake and return to their lane with sufficient time to complete the movement safely before an oncoming vehicle arrives. Road design guides contain many simplified methods to compute overtaking distance and these vary widely, especially at high design speeds. They are also generally impractical to achieve except on roads in flat or gently undulating country.

Experience has shown that safe overtaking manoeuvres can be performed with shorter sight distances and this has lead to the concept of Intermediate Sight Distance which is described in more detail in Section 2.9.5.

Overtaking sight distance is measured between two points 1.05 and 1.15 m above the road surface, ie. driver eye height and the height of an opposing vehicle.

2.9.5 Intermediate Sight Distance (ISD)

Observation of overtaking behaviour on two lane two-way roads has confirmed that drivers will attempt to overtake in situations where the sight distance available does not meet the distance calculated for the overtaking manoeuvre. It has been found that drivers will attempt to overtake:

- if there is a reasonable length of clear road ahead of them, and
- there is no oncoming vehicle in sight, and
- will break off the manoeuvre if an oncoming vehicle appears before the vehicle being overtaken is passed.

Analysis of the observed data indicated the circumstances in which drivers attempted to overtake when the sight distance available was well below the computed overtaking sight distance and, that when a distance equivalent to twice the stopping sight distance was available, overtaking could be attempted with reasonable safety. This sight distance has been defined as *'Intermediate Sight Distance'* and it is measured between two points 1.05 and 1.15 m above the road, ie. it is the distance needed for two opposing drivers travelling at design speed to safely stop their vehicles before colliding with each other.

Intermediate sight distance also makes it possible for a driver to see a 200 mm high object on the road ahead at a distance of approximately 1.4 times the normal stopping sight distance.

In practice, intermediate sight distance enables drivers to travel a road in comfort and at the same time provides many reasonably safe overtaking opportunities.

Table 2.13 shows intermediate sight distances which must be used in state highway road design. The speed used to determine intermediate sight distance must be not less than the design speed, or observed 85th percentile, speed.

2.9.6 Headlight Sight Distance (HSD)

(a) General

'Headlight Sight Distance' ensures that the roadway ahead will be illuminated by vehicle headlights for a distance equal to the stopping sight distance and it is a critical criteria in sag vertical curve design

The most common obstruction on a rural road at night is another vehicle, which may or may not be stopped. Even if it does not have its lights on it will have retro-reflective material at strategic locations which are situated higher than the object height used in the stopping sight distance calculation.

120 m to 150 m is the maximum sight distance that can safely be assumed for reasonable visibility of a stationary object on the roadway at night, ie. a safe stopping distance for 80 to 90 km/h. Beyond this distance only large or light-coloured objects will be perceived in time for a driver to take reasonable evasive action on an unlit road. Vehicle equipment therefore limits the design speed on unlit roadways to approximately 90 km/h.

The only method of achieving higher speed stopping sight distances on a road at night is the installation of an adequate level of street lighting.

The relatively small number of accidents involving objects on the roadway at night is probably due to:

- the factors of safety implicit in the various assumptions made in the idealised sight distance calculations, and
- the fact that most of the hazards likely to be met are other vehicles which are either illuminated or made visible by their legally required retro-reflective fittings.

Retro-reflective materials respond to much lower light levels than the non-reflective objects and are able to be perceived well outside the direct headlight beam. The provision of retro-reflective road furniture such as edge marker posts and reflective raised pavement markers are therefore important aids to drivers which help offset the shortcomings of vehicle lighting systems.

The assumptions made in the calculation of headlight sight distance are:

- vehicle headlights are mounted 750 mm above the road surface,
- the headlight beam is depressed 0.5° below the horizontal plane, and at best, provides some useful illumination up to the horizontal plane, and
- a zero object height.

Table 2.14 shows the headlight sight distances which must be used in state highway road design.

(b) Sag Vertical Curves

The effect of beam depression in a sag vertical curve reduces the effective headlight illumination distance. However, the length of a sag vertical curve necessary to provide headlight stopping sight distance is considerably more than that required to achieve a reasonable level of riding comfort. Refer to Section 2.10 for details of the effects of vertical acceleration on vehicle occupants.

Where a sag vertical curve is located within a horizontal curve vehicle headlights will shine tangentially to the horizontal curve and off the pavement. Although a horizontal headlight beam spread of about 3° left and right can be assumed, increasing the length of the sag curve to give the theoretical headlight stopping sight distance may not have the desired effect.

Design Speed *V* (km/h)	Coefficient of Longitudinal Deceleration *d*	Distance Travelled (m): Reaction Time *R* (sec) 2.0	Distance Travelled (m): Reaction Time *R* (sec) 2.5	Distance Travelled (m): Braking (m)	Rounded Stopping Sight Distance SSD (m)	
30	0.52	16.7	20.8	6.8	25	**30**
40	0.52	22.2	27.8	12.1	35	**40**
50	0.52	27.8	34.7	18.9	50	**55**
60	0.48	33.3	41.7	29.5	65	**75**
70	0.45	38.9	48.6	42.9	85	**95**
80	0.43		55.6	58.6	**115**	
90	0.41		62.5	77.8	**140**	
100	0.39		69.4	100.9	**170**	
110	0.37		76.4	128.8	**210**	
120	0.35		83.3	162.0	**250**	
130	0.33		90.3	201.6	**300**	

[1] ***May be used in urban areas and, with the appropriate approvals, in low speed rural areas.***

Table: 2.12: Stopping Sight Distance on a *LEVEL GRADE*

Design Speed (km/h)	Rounded Intermediate Sight Distance (m)
30	60
40	80
50	110
60	150
70	190
80	230
90	280
100	340
110	420
120	500
130	600

Table 2.13: Intermediate Sight Distance

Design Speed (km/h)	Rounded Headlight Sight Distance (m)
30	30
40	40
50	55
60	75
70	95
80	115
90	140
> 90	150

Table 2.14: Headlight Sight Distance

2.9.7 Application of Sight Distance Standards to Visibility on Horizontal Curves

Widening the inside of a cutting on a horizontal curve may be necessary to obtain the required sight distance. This will often mean the construction of a flat area or bench over which a driver can see an approaching vehicle, or an object on the road. In plan view, the extent of benching is fixed by the envelope formed by the lines of sight when:

- the driver and the object on the road are assumed to be in the centre of the inner lane, and
- sight distance is measured around the centre line of the lane, ie. the path a vehicle would follow when braking.

Where widening is not possible the horizontal curve radius must be increased to provide the sight distance necessary within the lateral clearance restriction imposed by obstacles such as cut batters, bridge abutments, etc. This in effect means that the minimum design speed horizontal curve radii is set by lateral clearance restrictions and not by the combination of superelevation and side friction. Short lengths of restricted sight distance must be identified and can only be approved by the Strategy and Standards Manager.

On multilane roads benching that provides sight distance for the inner lane traffic will more than meet the sight distance requirements for the outer lane.

Where a horizontal curve and crest vertical curve overlap, the line of sight between approaching vehicles may not be over the top of the crest but to one side of it, and may even be partly outside the road formation. Cutting down the pavement crest will not increase visibility when the line of sight is clear of the pavement and the bottom of the bench may need to be made lower than the shoulder level. In these situations, and also in the case of a sharp horizontal curve, a larger radius curve should be used to keep the line of sight within the road formation. This will however tend to increase the 85th percentile speed and hence the design speed.

2.9.8 Application of Sight Distance Standards to Two-lane Two-way Roads

Stopping sight distance, as measured from a driver eye height of 1.05 m to an object height of 200 mm, is the absolute minimum sight distance that must be provided wherever intermediate sight distance cannot be achieved.

Intermediate sight distance is the desirable minimum sight distance that should be provided on two-lane two-way roads if restrictive no-passing pavement markings are to be avoided. This will provide many passing opportunities even though it does not satisfy the stringent requirements assumed in the theoretical derivation of overtaking sight distances.

2.9.9 Application of Sight Distance Standards to Dual Carriageway Roads

Three sight distance requirements must be satisfied on a divided carriageway road. These are:

(a) Stopping sight distance at the design speed, as measured from a driver's eye height of 1.05 m to an object height of 200 mm, must be provided. However, wherever possible greater stopping sight distances should be provided.

(b) A minimum distance of 1.4 times the design speed stopping sight distance, as measured between two points 1.05 and 1.15 m above the road, ie. driver eye height to vehicle height, must also be provided.

This additional sight distance requirement is necessary because drivers on dual carriageway roads take some time to recognize that a vehicle ahead of them has stopped and, when it has, whether it is in a traffic lane or off the road on the shoulder. The application of this sight distance requirement often means that greater lateral clearances to obstructions on horizontal curves are needed, otherwise it is not normally as critical as stopping sight distance.

NOTE: ***The provision of overtaking and Intermediate sight distance is not necessary on multilane one-way carriageways.***

(c) At off ramps, sight distance measured from a point 1.05 m above the road to the pavement surface must be provided for drivers in the left lane of the through carriageway, ie. driver eye height to zero object height. This provides 1.4 times the design speed stopping sight distance and ensures drivers will have good visibility on the approaches to ramps, which will allow early identification of the ramp alignment and road markings.

2.9.10 Other Sight Distance Restrictions

Sight distance constraints which should also be considered during road design work include:

- Lines of trees which can restrict visibility at a sag because sight lines are interrupted by the foliage. A similar situation may occur where a bridge is located in a sag and lines of sight are cut by parts of the structure.
- Guardrail bridge handrails, median kerbs and similar obstructions which can restrict visibility at horizontal and vertical curves, and intersections located close to a bridge.
- Whether the curve ahead of a driver is to the left or right makes a large difference to the sight distance available.

2.10 Vehicle Occupant Ride Comfort (Vertical Acceleration)

A human being subjected to rapid changes in vertical acceleration feels discomfort. However, vertical acceleration only becomes critical in the design of sharp sag curves.

For normal state highway road design purposes the vertical acceleration generated when passing from one grade to another should be limited to a maximum of $0.05g$, where g = the acceleration due to gravity, ie 9.8 m/sec^2.

On low standard roads, and at intersections, a vertical acceleration of $0.10g$ may be used, where necessary.

The vertical component of acceleration normal to the curve, when traversing the path of a parabolic vertical curve at uniform speed is given by:

$$a = \frac{V^2}{12960\ K}$$

Where:

a = vertical component of radial acceleration (m/sec^2)

V = speed (km/h)

K = vertical curve parameter (m/1% change in grade)

K values for various design speeds and vertical accelerations are shown in Table 5.4.

2.11 Bridge Width and Structural Clearances

2.11.1 General

Refer to *Appendix A* of the *Transit New Zealand Bridge Manual* for full details of state highway bridge widths and structural clearance requirements. The most important points are summarised in Sections 2.10.2 and 2.10.3.

2.11.2 Bridge Width

(a) Bridge deck width is the sum of the carriageway width and the individual elements required to make up the desired bridge cross-section. A flowchart to aid in the determination of state highway bridge widths is shown in *Figure A3* of the *Bridge Manual*.

(b) The full width of the approach road traffic lanes and shoulders required for a road carrying the expected AADT 30 years ahead shall be provided across bridges of lengths shown in Table 2.15, except where:

- **(i)** the approach road is kerbed, or the bridge has a footpath(s), in which case the edge clearance selection criteria in 2.10.2 (c) (i) and (ii) below shall apply, or
- **(ii)** the approach road shoulder width is less than the edge clearance between the safety barrier and the adjacent traffic lane shown in Table 2.16. In these cases the clearance shown in Table 2.16 must be provided.

Road Type	Bridge Length (m)
Dual Carriageway:	
Motorway	≤ 75
Expressway	≤ 30
Two-lane two-way road where the AADT is:	
> 4000	≤ 30
2000 - 4000	≤ 15
500 - 2000	≤ 9
< 500	≤ 6
NOTE: AADT is the expected traffic volume 30 years ahead.	

Table 2.15: Length of Bridge Requiring a Full Carriageway Deck Width

(c) In all other situations bridge carriageway width shall to be determined by the following method:

- **(i)** **Traffic lane(s):** The full width of the approach road traffic lanes, including any extra widening required for heavy vehicle tracking paths on small radii horizontal curves, must be provided on the bridge in all cases.
- **(ii)** Where the approach road is kerbed, the bridge kerbs shall be aligned with the approach road kerbs. The bridge carriageway width is therefore the same as the approach road width.
- **(iii)** Where the approach road is not kerbed but the bridge is (usually a footpath kerb), a minimum clearance of 600 mm from the kerb face to the edge of the adjacent traffic lane must be provided. The bridge carriageway width is the sum of the approach road traffic lane widths plus the edge clearance widths.
- **(iv)** Where neither the approach road or the bridge are kerbed a clearance (shoulder) between the safety barrier and the edge of the adjacent traffic lane must be provided in accordance with Table 2.16. The bridge carriageway width is the sum of the approach road traffic lane widths plus the edge clearance (shoulder) widths.

Road Type	Minimum Clearance (mm)
(a) Low volume one-lane or two-lane roads (AADT < 500)	600 generally (300 absolute minimum)
(b) Medium volume two-lane roads: (i) AADT 500 - 2000	750 generally (600 absolute minimum)
(ii) AADT 2000 - 4000	1000 generally (600 absolute minimum)
(c) High volume two-lane roads (AADT > 4000)	1200 generally (600 absolute minimum)
(d) Dual carriageway roads (Motorways / Expressways)	1200 generally (600 absolute minimum)

NOTES:
1. ***AADT is the expected traffic volume AADT 30 years ahead.***
2. ***General minimum clearances shall apply unless there are compelling reasons to use lesser clearances. Absolute minimum clearances should only be used in extreme conditions, e.g. where it is physically impracticable to provide the normal clearance.***
3. ***For cycle facilities refer to the Bridge Manual Appendix A: A1 (c).***

Table 2.16: Clearances between Bridge Safety Barriers and the Edge of the Adjacent Traffic Lane

2.10.3 Structural Clearances

(a) General

The minimum horizontal and vertical clearances for structures over state highways are shown in Figure 2.13.

(b) Provision for Over Dimension Loads

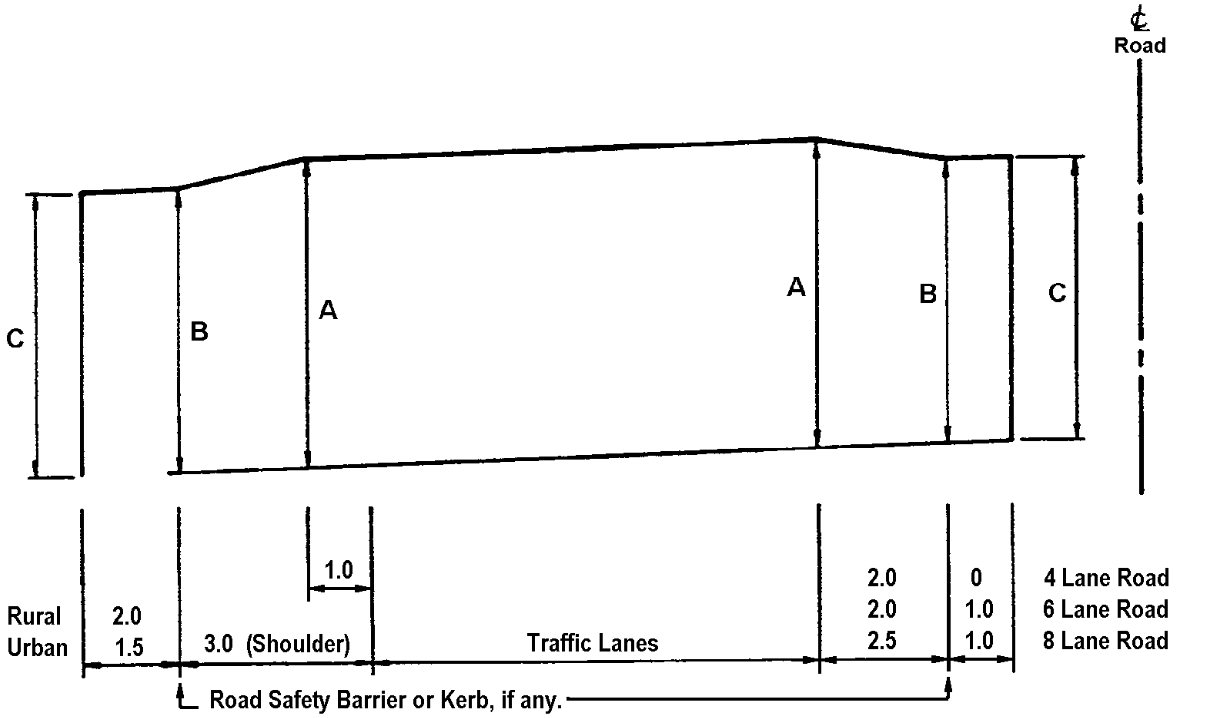
(i) Structures crossing state highways should not normally cater for over dimension loads unless:

- it is economically viable to build the structure to provide the over dimension clearances noted in (ii) below, and
- all other structures on that section of state highway also provide the over dimension clearance envelope, ie. the state highway forms a consistent over dimension route.

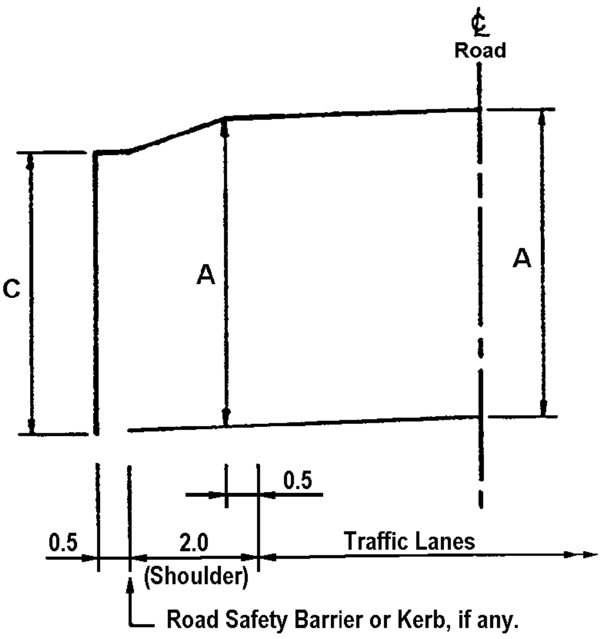
(ii) Where a state highway forms part of an official over dimension route all structures crossing the road must provide a horizontal clearance at least 10.5 m and a vertical clearance of at least 6.0 m.

(c) Overhead Signs and Traffic Signals

All signs and traffic signals located over the road carriageway way lanes must have a minimum vertical clearance of 5.4 m.



(a) Divided Roads



(b) Two-lane Two-way Roads

Notes:

1. Where the Transit New Zealand Authority has resolved that a state highway, or a section of a state highway, is an Overimension Load Route, all new structures crossing that section of road shall provide a minimum vertical clearance of 6.0 m over a carriageway width of at least 10.0 m, to allow the passage of overdimension loads.
2. All overhead clearances shall be measured vertically.
3. Lateral clearances shall also be checked for sight distance on curved alignments.
4. Overhead clearances to footpaths shall be as large as possible, but not less than 2.5 m.
5. Vertical Clearances at Structures:

DIMENSION	Absolute Minimum (m)	Preferred Minimum * (m)
A	4.9	6.0
B	4.5	5.6
C	4.5	5.6

* ***Minimum for Overdimension Load Routes. May be used on other routes, when economically justified.***

6. Design vertical clearances shall make provision for settlement and road surfacing overlays. As a guideline, provide a minimum of 100 mm more than the design vertical clearance where an overlay is anticipated.
7. Vertical Clearances at Pedestrian Bridges:
   - **(a)** At least 200 mm greater than adjacent traffic bridges, but not less than 5.1 m.
   - **(b)** At least 6.2 m when there are no adjacent traffic bridges.

Figure 2.13: Minimum Horizontal and Vertical Structure Clearances for State Highways