TRANSPORT RESEARCH LABORATORY

Accidents on modern rural single-carriageway trunk roads

Prepared for Traffic, Safety and Environment Division, Highways Agency

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TRL REPORT 336

First Published 1998
ISSN 0968-4107

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Executive Summary

In 1991 there was 5,300 km of single-carriageway trunk road in England, and the expanded roads programme announced in 1989 contained proposals for a further 400 km of new single-carriageway trunk road (though these have since been reviewed).

The layout standards for the construction of new roads and the improvement of existing roads have been developed by the Department of Transport (now the Department of the Environment, Transport and the Regions, DETR) over many years, and this process of development is still continuing. The main objective has been to design roads that satisfy three important criteria: to provide high traffic capacity and low delay; to reduce road accidents; and to achieve these with low land acquisition, construction and maintenance costs.

This study of single-carriageway trunk roads of modern design aims to provide information which will help achieve the second of these objectives: to reduce road accidents. It quantifies the relationships between the numbers of injury accidents that occur, and the traffic and road layout variables that determine them. The study was commissioned by the Traffic, Safety and Environment Division of the Highways Agency and was undertaken by the Traffic and Transport Resource Centre of TRL.

The study was based on 103 new schemes, totalling 540 km in length, which comprised most of the new non-built-up single-carriageway trunk road schemes in England which opened between 1968 and 1989. The accident period studied was from 1979 to 1990. There were 2502 accidents during this period on the schemes studied, of which 1295 were link accidents, 391 occurred at major junctions and 816 at minor junctions. The main results of this study refer to links, that is, to the stretches of road between junctions.

The techniques of generalised linear modelling were used to develop predictive models of the annual numbers of accidents on a length of road in terms of the explanatory variables that affect them. These variables were the traffic flow, the length of the road and the features of the road such as carriageway width and the presence of kerbs and hardstrips. The study also examined trends in accident numbers and rates over time.

Tabular analysis of accident rates was used in order to provide a preliminary overview of the main variables which affect accidents, and to give a lead as to which variables should be included in the regression modelling. The report presents results from this tabular analysis regarding the effect on accident rates of carriageway width and the presence of kerbs and hardstrips.

The main results of the study can be summarised as follows:

- Modern single-carriageway trunk roads are safer than traditional A-roads, having a little over half the accident rate. Therefore, replacing older roads with new ones will lead to a reduced accident risk.
- The newest modern trunk roads, opened since the mid-1980s, are safer than those built earlier. The difference is due to the more widespread use of wide carriageways, hardstrips and other road design features which contribute to fewer accidents.
- Roads with hardstrips are safer than those of the same width without hardstrips, having about 20 to 25 per cent fewer accidents, all else being equal.
- Wider (WS2) road schemes with 10 metre carriageways are safer than standard-width (S2, 7.3m) schemes, having 27 per cent fewer accidents, all else being equal.
- Uphill gradients lead to slightly fewer single-vehicle accidents but horizontal alignment was not found to have any significant effect on accident risk.

1 Introduction

1.1 Background

In 1991 there was 5,300 km of single-carriageway trunk road in England (Department of Transport, 1992). The expanded roads programme announced in 1989 (Department of Transport, 1989a) contained proposals for a further 400 km of new single-carriageway trunk road, though in following reviews of the programme not all of these proposals are proceeding.

The layout standards for the construction of new roads and the improvement of existing roads have been developed by the Department of Transport (DOT, now the Department of the Environment, Transport and the Regions) over many years, and this process of development is still continuing in the Highways Agency. The main objective has been to design roads that satisfy three important criteria: to provide high traffic capacity and low delay; to reduce road accidents; and to achieve these with low land acquisition, construction and maintenance costs.

The purpose of the study presented in this project report is to quantify the relationships between the numbers of injury accidents that occur, and the traffic and road layout variables that determine them, for single-carriageway trunk roads of modern design. The study was restricted to non-built-up single-carriageway trunk roads in England which were new schemes opened since 1968.

The study was commissioned by the Traffic, Safety and Environment Division of the Highways Agency and was undertaken by the Traffic and Transport Resource Centre of TRL.

TRL has carried out a similar study of the relationship between accidents and road design on modern rural dual-carriageway roads (Walmsley, Summersgill and Payne, 1998), and where appropriate in this report comparisons are drawn between the two studies. Further work on the relationship between road layout and accidents on single- and dual-carriageway rural trunk roads is reported in Walmsley and Summersgill (1998).

1.2 Study objectives and approach

The main objectives of the study were:

- to identify those layout variables that have an effect on accident risk on modern rural single-carriageway trunk roads and to quantify the effect;
- to investigate the effect of vehicle speed on accidents;
- to investigate the extent to which accident rates are changing over time.

The approach used was to conduct a cross-sectional study of most modern rural single-carriageway trunk road schemes in England. Data relating to these schemes were obtained from a variety of existing sources in order to avoid the substantial costs of on-site data collection. STATS19 accident records were obtained for all relevant injury accidents since 1979, so as to maximise the number of accidents available for study and to determine the trends in accident risk. The initial analysis was based on tabulation of the data, and the later analysis used the statistical regression methods of generalised linear modelling.

1.3 Contents of the report

1.3.1 Layout standards

Modern trunk roads are built to a design known as Highway Link Design (HLD), specified in the Highways Agency Design Manual (Highways Agency, 1997a) for the layout of many geometric features of a new road. The main features of HLD are summarised in Section 2.

1.3.2 Literature survey

A review of previous accident studies on single-carriageway roads was conducted to assist in determining the variables that would be most likely to affect accident risk. Details are presented in Section 3.

1.3.3 Scheme selection and subdivision

For the purposes of modelling accidents on single-carriageway rural trunk roads, a sample of sites was selected comprising all single-carriageway trunk road schemes which opened between 1968 and 1989. The original sample contained 111 schemes, but it was necessary to eliminate 8 of these due to non-availability of data, leaving 103 schemes for analysis.

Each scheme was divided into major links, that is, stretches of road between major junctions, which are defined as junctions where traffic on the major road has to give way (such as at roundabouts or traffic signals). Major links were subdivided into minor links by the occurrence of minor junctions, which are any other properly marked junctions where traffic on the major road has priority. The minor link is the basic unit for analysis in this study. The sample of schemes contained 429 minor links. Details are presented in Section 4.

1.3.4 Data

Four types of data were extracted for these schemes: accidents, traffic flows, alignment variables (curvatures and gradients) and design variables (geometric measurements such as verge widths and presence of features such as kerbs and hardstrips).

Accident data were supplied by Safety and Environment Resource Centre, TRL. The data related to all accidents on the roads concerned between 1979 and 1990. These data were supplied to an external contractor who performed two tasks. Firstly, they allocated each accident to one of 67 accident types, and these were further aggregated into 24 groups. Secondly, they located each accident on a map and assigned it to the appropriate minor link or junction. There were 2502 accidents in the data set, of which 1295 were link accidents, 391 occurred at major junctions and 816 at minor junctions. Section 5 gives further details.

The factors that were taken into account in selecting some of the variables that were measured are discussed in Section 6.

Traffic flow data were supplied by Statistics Transport C Division, Department of Transport, from their rotating traffic census, for census points covering the schemes of interest. The data covered the years 1983 to 1990, but no data were available for some census points; these gaps were filled by requesting data from County Councils and DOT Regional Offices. Details are presented in Section 7.

Alignment data were obtained from a survey of all trunk roads carried out for DOT using a machine known as the High Speed Road Monitor (HRM). Developed at TRL for monitoring road surfaces, this device measures curvature, gradient and road texture at 10 metre intervals, thus providing the study with road alignment measures in much greater detail than could be achieved with the more usual technique of map measurement. The data were provided by the contractor who carried out the original survey for DOT, and were processed by TRL to produce measures of bendiness, hilliness and other alignment measures for each minor link. A full description is presented in Section 8.

Other design variables were obtained from video tapes, using videos from a survey of all trunk roads carried out for DOT as part of the Network Information System. The videos were observed and analysed, and measurements or estimates were made of a large number of variables as specified by TRL. Link data were obtained at intervals of approximately 75 metres along each scheme, and junction data at every major and minor junction on each scheme. Variables relating to whole minor links for the later statistical analysis were constructed using the 75-metre data as a basis. Further details are given in Section 9.

The methods by which variables for carriageway width and the presence of kerbs and hardstrips were derived are described in Section 10. The derivation of estimates of the speed of vehicles on each link, based on formulae used in cost benefit assessments for new highways (Department of Transport, 1981, and Lee and Brocklebank, 1993), is also described in Section 10.

1.3.5 Analysis and conclusions

Accident tabulations to investigate the characteristics of the accidents are presented in Section 11.

Tabular analysis of accident rates was used in order to provide a preliminary overview of the main variables which affect accidents, and to give a lead as to which variables should be included in the regression modelling. These results are presented in Section 12.

A cross-sectional analysis of the data using the techniques of generalised linear modelling with the GENSTAT (Alvey *et al*, 1977) statistical package was used to develop mathematical models that relate the numbers of accidents to the traffic flow and design features of the road. An investigation of trends in accident frequencies over time is presented in Section 13. The methodology of the regression analysis is described in Section 14. Total accident-flow models for minor links, and similar models for individual groups of accidents, including the effect of a number of design variables, are presented in Section 15.

A summary of the model results and their implications for the existing standards are discussed in Section 16, and the main conclusions from the study are summarised in Section 17.

2 Road layout standards

2.1 Layout of Roads in Rural Areas (LRRA)

Simpson and Kerman (1982) give a brief history of the development of road design standards by the Department of Transport and its predecessors, from 1937 to the early 1960s. The principle behind these standards was to provide easy curves and gradients and to maximise sight distances as far as possible. The culmination of this development was the publication of comprehensive design standards in Layout of roads in rural areas (LRRA) (Ministry of Transport, 1968).

2.2 Highway Link Design (HLD)

More recently, the standards have been completely revised to produce a comprehensive standard known as Highway Link Design (HLD) (Highways Agency, 1997a). The new design addresses two problems with the earlier designs. First, in some circumstances it is uneconomic to design gentle curves when with a sharper curve it may be possible to avoid obstacles and reduce land take. Second, the long curves of LRRA design can produce conditions where high speeds are possible but where overtaking is hazardous, even though it can take place almost everywhere.

The objective of Highway Link Design is to maximise safe overtaking opportunities by building roads with fairly straight sections which are linked by sharper bends, providing a clear indication to the driver when overtaking is, or is not, safe. In this way, it is often possible to design roads at less cost, because the requirement for land is less than for the earlier LRRA design which required long, sweeping curves.

The principle behind Highway Link Design (Highways Agency, 1997a) is that there is a margin below the desirable minimum standards where safety is not significantly reduced. It is therefore permissible, in places where it would be uneconomic to maintain the standard, to accept relaxations in the standard without compromising safety, provided drivers can perceive (and are advised by signs etc) that there is a potential hazard.

HLD specifies design speeds for new roads in a series of steps, namely 50, 60, 70, 85 and 100 km/h (plus 120 and 145 km/h on dual carriageways and motorways). The Design Speed sets the standard for a whole range of other features of the road such as bends and gradients. These are normally built to a Desirable Minimum Standard, using criteria such as maximum sideways acceleration on bends. In some cases, where it is more economical to do so, the standards allow some features of the road to be designed for one or more speed steps less than the design speed. In certain circumstances, where justified, the Highways Agency can authorise a Departure from standard.

With regard to bends, research quoted in Simpson and Kerman (1982) (see for example Shrewsbury and Sumner, 1980; Shepherd and Lowe, 1982) has shown that sharp bends increase accidents to only a moderate extent, because drivers can perceive a bend to be hazardous. Accordingly, they do not negotiate it at the design speed, but slow down and avoid overtaking. In addition to the specification of a desirable minimum radius, the HLD standards for bends are therefore formulated so that bends are either gentle enough to be negotiated without speed reduction or loss of overtaking sight distance, or sharp enough to be clearly perceived and marked as a hazard. The former are designated as Range A or B curves, and correspond to radii of greater than 8160 metres, and between 2880 metres and 8160 metres, respectively, for a design speed of 100 km/h. The latter are designated as Range D curves, and correspond to radii between 510 metres (the Desirable Minimum radius) and 1020 metres. Intermediate radius bends are avoided. These are designated as Range C, and for a design speed of 100 km/h correspond to radii between 1020 metres and 2880 metres.

3 Literature survey

A review of the literature was made to identify variables that had been tested in previous studies of accidents on single-carriageway rural trunk roads, both all-purpose and motorway, and to see which variables had been shown to have an effect on accidents.

3.1 Previous accident studies on UK single-carriageway rural trunk roads

An unpublished study was carried out by MVA to identify accident rates on LRRA and HLD-like roads. The study was, however, carried out before any true HLD roads were open; it therefore compared stretches of existing road which were similar in design to HLD. The study concluded that for roads with kerbed edges LRRA-designed roads had a lower accident rate than HLD-like roads, though for roads with metre-strip edges (as specified in true HLD) there was no significant difference. Roads with metre-strip edges generally had lower accident rates than kerbed roads, and wider (10m) roads were safer than normal-width (7.3m) roads.

A further brief study of accident rates on all English rural trunk roads was carried out by DOT. The results for single-carriageway roads are of interest in the present study. For the accident period from 1981 to 1984, the injury accident rate for traditional older designs with kerbs, at about 0.20 PIA per million vehicle-kilometres, was higher than for 7.3m roads with a metre strip and designed and built since the 1970s (0.16 PIA per MVkm). The injury accident rates included accidents on the links and at those junctions at which the main route had priority (minor junctions). For roads with metre strips designed and built since the 1970s, the rate during the period from 1987 to 1990 (0.22 PIA per MVkm), was higher than in an earlier period from 1981 to 1984 (0.16 PIA per MVkm). It is not clear from this result whether the increase in accident rate was attributable to a general increase in accident rates on these roads, or whether the accident rate on those roads opened most recently was higher than on the somewhat older roads. In the period from 1987 to 1990, the accident rate on wider 10m roads with a metre strip (0.16 PIA per MVkm), was lower than on the 7.3m roads with a metre strip (0.22 PIA per MVkm).

3.2 General survey

A literature search was made to identify previous studies which investigated the effects of the geometric and other variables of rural single-carriageway road links on accidents. Some of the studies were not confined to rural single-carriageway links, but also investigated junction accidents on motorways and dual carriageways. Some of the studies were on British roads (Shrewsbury and Sumner, 1980; McBean, 1982; McGuigan, 1982; Shepherd and Lowe, 1982; Silcock and Worsey, 1982; Simpson and Kerman, 1982; Simpson and Brown, 1985). Some were studies in other countries, the results of which might or might not be applicable to British roads (Lundy, 1965; Cribbins *et al*, 1967; Kihlberg and Tharp, 1968; Dart and Mann, 1970; Banks and Brown, 1979; Wass, 1983). Some were specific studies of the effect of just one characteristic such as traffic flow or lane width on accidents (Zegeer *et al*, 1981; Maltby and Bennett, 1986; Turner and Thomas, 1986; Blakstad, 1987). Some were themselves reviews of earlier work in the field (Silyanov, 1973; Satterthwaite, 1981; MacLean, 1985).

Most of the studies made use of existing data - national or state databases and the like; very few made on-site measurements. The results of this search are summarised in the following paragraphs.

3.2.1 Traffic flow

Most authors studied the effect of traffic flow on accident rates (per vehicle-km), and some only studied this variable. The results were surprisingly variable. Some studies (Silcock and Worsey, 1982; Turner and Thomas, 1986; Blakstad, 1987) found no significant effect, others (Cribbins *et al*, 1967; Kihlberg and Tharp, 1968; Silyanov, 1973; Satterthwaite, 1981; Zegeer *et al*, 1981; McGuigan, 1982; Wass, 1983) found a significant effect, though its magnitude and direction (whether accident rates increase or decrease with increasing flow) varied.

All authors who tested the effect on accident rates of the proportion of heavy goods vehicles in the traffic flow found the effect significant.

3.2.2 Road design standards

As noted earlier, the unpublished MVA study found significant differences between accident rates on LRRA, HLD-like and traditional roads.

3.2.3 Accident severity

Only one study (MVA, unpublished) found that accident severity was related to geometry; results from other studies were inconclusive.

3.2.4 Number of vehicles involved

There was considerable evidence (Kihlberg and Tharp, 1968; Satterthwaite, 1981; Zegeer *et al*, 1981) that the effect of traffic flow on single- and multi-vehicle accident rates was different, the former decreasing, and the latter increasing, with increasing flow. The variability of findings on the effects of traffic flow on total accident rates (section 3.2.1 above) is probably due to differing proportions of these two effects. In some studies, these categories were broken down according to whether a single vehicle ran off the road or not, and whether vehicles in multi-vehicle accidents were travelling in the same or opposite directions.

3.2.5 Alignment

Horizontal alignment was found to have a highly significant effect on accident rates in almost every case. One study (Shrewsbury and Sumner, 1980) found a difference in accident rates on left and right hand bends but another (McBean, 1982) found none.

Most studies found gradient significant, though to differing degrees, and in one study the effect of gradient was found to be flow-dependent. There was evidence that accident rate increased with gradient for slopes up to about |4| per cent, but increased less rapidly for steeper gradients.

Two studies (Silyanov, 1973; Shrewsbury and Sumner, 1980) found a difference between uphill and downhill gradients, and the latter also found an effect of vertical curvature, with a steepening downhill gradient being significant, though increasing uphill gradient was not. No other studies examined this effect.

Five studies (Silyanov, 1973; Shrewsbury and Sumner, 1980; Satterthwaite, 1981; McBean, 1982; Shepherd and Lowe, 1982) tested the effect of sight distance, and found it significant, though to varying degrees - Silyanov (1973) and McBean (1982) found it was significant only up to 200-300 metres. There are some doubts about the validity of these significances since sight distance is strongly correlated with curvature.

3.2.6 Cross-section

Nearly all authors found a significant reduction in accident rates as carriageway width increased, but with some evidence (Silyanov, 1973; Wass, 1983) that increases beyond around 8m had little effect. The number of lanes (which was correlated with width) was also significant, though less so. The width of the verge was found to be significant in three studies (Dart and Mann, 1970; Silyanov, 1973; Banks and Brown, 1979), less so in another two (Zegeer *et al*, 1981; MacLean, 1985), and insignificant in another (Shepherd and Lowe, 1982). The type of verge (grass/gravel, footpath, wall, ditch etc) was not found to be significant in the two UK studies (McBean, 1982; Shepherd and Lowe, 1982) which examined it, though there was strong evidence from US studies that the width, slope and occurrence of obstructions on the verge were important.

One study (MVA Consultancy, 1986) found that roads with 1-metre hard strips had lower accident rates than continuously kerbed and other edges. Another (McBean, 1982) found no effect. Only one study (Shepherd and Lowe, 1982) tested edge markings, and did not mention them as significant.

3.2.7 Other variables

Most authors found that the number of junctions and accesses, or their presence or absence, had a significant effect on accident rates, though to a varying degree.

Land use alongside the road, categorised as ‘undeveloped’ (open or woodland) or ‘developed’ (buildings), was found to have a significant effect on accident rates in some studies (McGuigan, 1982; Shepherd and Lowe, 1982; Silcock and Worsey, 1982), but not in another (McBean, 1982).

Site length was a strong determinant of accident frequency (number of accidents per annum) along a length of road. Some studies had tested whether there was an effect of site length on accident density (accidents per km) or accident rate (accidents per vehicle-km), that is, whether the accident frequency was non-linearly dependent on length. Two studies (Kihlberg and Tharp, 1968; Maltby and Bennett, 1986) found an effect on accident rate, though the unpublished MVA study cited earlier found no significant effect.

Accident rates were found to depend on year (Shepherd and Lowe, 1982), and in the UK case, were different before and after the introduction of seat-belt legislation in 1983.

In some studies (Cribbins et al, 1967; Silyanov, 1973; Blakstad, 1987) speed limit had a significant effect on accident rates, but the results were inconsistent.

The effect of signs and markings was not significant (Cribbins et al, 1967; Shepherd and Lowe, 1982), except in the case of double centre lines (McBean, 1982), though here there was a correlation with bends.

The effect of geometric variables on accidents in daylight or darkness, or by month, or in different weather conditions, was not found to be different in the studies which tested them (Cribbins et al, 1967; Dart and Mann, 1970; Shrewsbury and Sumner, 1980; Shepherd and Lowe, 1982).

The following variables were tested in at least one study but were not found to have a significant effect on accident rates:

- skid resistance (McBean, 1982; Shrewsbury and Sumner, 1982);
- superelevation (Dart and Mann, 1970; McBean, 1982; Shrewsbury and Sumner, 1982);
- the presence of roadside obstacles (Dart and Mann, 1970; McBean, 1982);
- the presence of merges/diverges and weaving areas (Shrewsbury and Sumner, 1982);
- climbing lanes;
- pedestrian facilities (Silcock and Worsey, 1982);
- the features of preceding sections of road (Shepherd and Lowe, 1982).

4 Scheme selection

4.1 Identification of modern single-carriageway schemes

The objective of the study was to investigate accidents on rural single-carriageway roads which had been designed and constructed in recent years, that is, under the LRRA or HLD design standards. It was therefore decided that the study would cover all single-carriageway trunk roads in England opened since 1968, to cover the whole period during which LRRA or HLD standards were in force, although in the earlier years there was some risk of including some pre-LRRA schemes. The study was restricted to trunk roads because of the limited availability of data for local authority roads.

A comprehensive list of single-carriageway road schemes (excluding minor improvements) was obtained from successive Roads White Papers (Ministry of Transport, 1969, 1970; Department of the Environment, 1972a, 1972b, 1974a, 1974b, 1976a, 1976b; Department of Transport, 1978, 1980, 1982, 1983, 1985, 1987, 1990). This gave an initial list of 128 single-carriageway schemes opened between 1 April 1968 and 31 December 1989. The lists for earlier years (1968-75) did not distinguish between single- and dual-carriageway roads, so it was necessary to identify single-carriageway schemes from maps.

Schemes from the resulting list were located on Ordnance Survey maps. Some of the schemes were eliminated on the grounds that they were junction improvements, bridges or viaducts, or short approach links to motorways. All schemes in Greater London, and some in other major cities, were also excluded on the grounds that they were not rural. This resulted in a definitive list of 110 schemes.

For the purposes of extracting data from videos and other sources, it was necessary to define precisely the stretch of road concerned. Accordingly, the start and end points of each scheme were marked on a set of site definition maps.

During the video viewing by the contractors, some schemes were found unsuitable and these were eliminated from the study. A list of the schemes is presented in Appendix 1.

4.2 Definition of units for study

This study was mainly concerned with accidents on links, that is, stretches of road between junctions. The following terminology is used in this study:

Major junction: where traffic on the major road has to give way or stop, usually at a roundabout or traffic signal, or where the road joins a more important road at a priority junction;

Minor junction: any other properly-marked junction where traffic on the major road has priority;

Access: an unmarked junction with a non-public road such as an entrance to a lay-by, factory, farm, driveway, track, private house, private drive, filling station, etc;

Major link: road between major junctions which may contain one or more minor junctions or accesses and one or more minor links;

Minor link: road between minor junctions (or a major and a minor junction) which may contain one or more accesses.

Junction length: for single carriageways, junctions were regarded as having zero length. This was different from the dual-carriageway study (Walmsley, Summersgill and Payne, 1998), where a junction could extend over a considerable distance.

Link length: the length of a minor link, or the total length of the minor links on a scheme.

Major link length: the length of a major link, or the total length of the major links on a scheme.

Scheme length: the total length of a scheme, equal to the sum of the link lengths.

Link accidents: accidents which occur on the scheme road but not within 20 metres of a junction.

Junction accidents: accidents which occur on the scheme road at or within 20 metres of a junction.

Some accidents which STATS19 classes as junction accidents (access accidents, for example) would be classed as link accidents according to these definitions.

Minor links were chosen as the units for the regression analysis. Some accident rates were also calculated for the junctions and for major links and whole schemes.

5 Accident data

5.1 Extraction of accident data

TRL has a well-established system for extracting accident data from STATS19 records, and this was used as the source of accident data for the study. Complete data were extracted for all years from 1979 (the earliest available year) to 1990.

For the purpose of extracting accident data, the 110 schemes in the definitive list were specified by an Ordnance Survey grid reference box. This was drawn with a generous margin of about 2 km at either end of the scheme, in order to avoid excluding any accidents which lay close to, but (due to an error of coding on the STATS19 form) not exactly on, the scheme. The accidents were extracted by the Safety and Environment Resource Centre of TRL, who checked to ensure that none had been missed, using their previous experience of various likely errors.

The data extracted from STATS19 records were:

a Variables giving the location of the accident (grid reference, road number, second road number at junction), for allocating the accident to the appropriate minor link.

b Details of vehicle manoeuvres from the vehicle records, required for classifying accidents into types and groups according to numbers of vehicles involved and manoeuvres.

c Details of the other attendant circumstances - consisting of most of the remaining fields from the STATS19 accident record.

d The numbers and types of vehicles, the severity of casualties, and the number of pedestrian casualties, summarised from the vehicle and casualty records.

There were several instances of missing data for some counties in certain years. All schemes for which the accident frequency in a particular year was unusually low or high were carefully investigated.

5.2 Location of accidents

Data on 12705 accidents were obtained from STATS19, and were passed to the external contractors for processing.

The first task was to eliminate accidents which occurred before or during the year in which the scheme concerned opened, and those occurring on schemes which were removed from the analysis by the contractors because of missing videos, roadworks etc. This reduced the total number of accidents from 12705 to 5248.

The remaining accidents were plotted on maps of individual schemes to identify where each occurred, and allocated to the appropriate link or junction within the scheme structure. During this process a number of accidents were found to occur at points other than on the scheme concerned. These could be broadly divided into: 830 accidents which occurred on the ‘old road’ which had been superseded by the scheme; 1169 accidents which occurred on the new road, but beyond the limits of the scheme under consideration (mainly due to the 2 km error margin in specifying grid references); 391 accidents which occurred on another road; 335 accidents which did not fall on any road, or had unexplained coordinates; 19 accidents which occurred on schemes which were eliminated due to missing alignment data. As a result, 2502 accidents remained for analysis.

Each remaining accident was assigned to a direction of travel, according to a precise scheme (Walker and Lines, 1991) for deciding which vehicle caused the accident - for example, if one vehicle was overtaking another, the direction of the accident was the direction of the overtaking vehicle. This information was needed in order to correlate accidents with some design features which depended on direction, for example visibility.

5.3 Allocation of accidents to types and groups

The accidents were assigned an accident type according to a classification which was developed according to three criteria: the number of accidents of that type, how easy it was to identify accidents of that type, and the subjective importance of separately identifying accidents of that type.

This classification resulted in 67 accident types (28 for junction accidents and 39 for link accidents). These were combined into 24 groups (9 for junction accidents and 15 for link accidents). The groupings mainly involved combining the 2- and 3-or-more- vehicle categories (since there were few accidents involving 3 or more vehicles), combining all pedestrian accidents into one group, combining some turning manoeuvres (especially left turns) into groups, and not distinguishing the points of impact in rear-end and head-on collisions. The accident types are listed in Appendix 2 and the groups in Appendix 3.

A small number of accidents were not classified into groups by the contractor because of the difficulty in analysing the manoeuvres. TRL made a detailed examination of the accident record which in most cases allowed them to be assigned to an appropriate group. In doing so it was found necessary to form an additional group Z, to take account of single-vehicle accidents where overtaking was involved (the second vehicle in the manoeuvre not being involved in the accident).

5.4 Allocation of accidents to classes

In view of the low numbers of accidents in many of the groups, for some purposes accidents were further grouped into accidents involving 1 vehicle and accidents involving 2 or more vehicles. These groupings are referred to as classes. The single-vehicle class consisted of groups L to R and Z, and the multi-vehicle class consisted of groups T to X. There were 338 single-vehicle accidents and 808 multi-vehicle accidents. Accidents which involved pedestrians, accesses or parked vehicles were not allocated to any class.

6 The selection of explanatory variables

The list of variables, both basic and derived, that were measured and assembled as possible candidates for the analysis are presented in Appendix 4. A number of factors were taken into consideration in choosing these variables.

Firstly, the overall purpose of the study was to identify areas where the design standards might be revised. With this in view, it was decided that the study should include those variables which are important parts of the standards; they were, in any case, variables which intuitively would be expected to have an effect on accident rates. The main ones were traffic flow, speed estimate, curvature, gradient, visibility, carriageway width, presence of kerb and presence of hardstrip.

Secondly, a review of the available literature was made to see what variables had been used in similar studies in the past. Most of the variables which other researchers had tested were obtained for this study. A summary of the literature search is presented in Section 3.

Thirdly, it was necessary to consider the available sources of data. In some cases, variables which would not be expected to have an effect, or which other researchers had found to be insignificant (such as type of edge marking), could readily be observed in the course of measuring other variables. It was also possible to think of a large number of other variables which might have an effect on accidents. These additional variables were recorded where they could be measured without undue effort. It will be clear that the study concentrated on assembling an extensive, low cost, yet reliable set of data for modern English non-built-up single-carriageway trunk roads.

The following sections discuss the variables which were obtained, the methods for obtaining them and the sources of data used.

7 Traffic measurements

Almost all accident studies agree that traffic flow is a key factor determining the number of accidents on a particular road. It was therefore essential to include traffic flow in the models.

7.1 Obtaining traffic flow data

Traffic data were mainly obtained from Statistics Transport C Division, DOT. STC maintain a rotating census of traffic flows on all main roads, covering 12000 census points over a 6-year cycle. The counts made are 12-hour counts on a weekday, and are grossed up to annual average daily traffic flows (AADTs). As each census point is counted only once every six years on average, STC generate the data for the intervening years using growth factors based on national traffic trends on roads of each type.

The data supplied by STC comprised AADTs for the census points requested for each year from 1983 to 1990 (the latest available). The data covered 11 vehicle types, namely Pedal Cycles, Cars, Motor cycles, Buses, Light Goods Vehicles, Heavy Goods Vehicles (broken down into 6 axle categories), plus totals of all vehicles and heavy goods vehicles.

There was no indication of which years' data were based on actual counts and which were projections. The data were therefore regarded as the best DOT estimate, however obtained, of the traffic flow for each Census Point in the year concerned.

STC data were not available for some schemes and therefore the relevant County Councils and DOT Regional Offices were approached for traffic counts. This produced a high level of response, and in most cases it was possible to assimilate the data into the form required.

When completed, the traffic data were assembled into a file containing AADT flows for each minor link, standardised for the year 1990, for 6 vehicle types (the HGV categories being amalgamated into one vehicle type).

7.2 Traffic growth

An estimate of the average traffic growth across all the schemes was obtained by fitting a log-linear regression line to the available traffic data. This gave an average growth rate over all schemes of 4.5 per cent per year over the period from 1983 to 1990, with a standard error of 0.3 per cent, giving a 95 per cent confidence interval from 4.0 to 5.1 per cent. The regression parameters were then used to provide an estimate of what the base-year (1990) traffic flow on each scheme would have been if the growth had been smooth rather than subject to year-by-year variation.

To some extent, the growth factors obtained would reflect the average of the national trends used by STC in estimating traffic flows for years when there were no actual counts. However, as most schemes would be counted at least twice during the period 1983-92 the growth factors obtained would also reflect the actual growth in traffic on the scheme concerned.

8 Alignment data

8.1 The importance of alignment measures

The term 'alignment data' is used here to denote details of the curvature and gradient of the road schemes in the study. Although these variables can be considered as geometric variables, they warrant separate discussion for three reasons. Firstly, other studies have shown that alignment variables are likely to be among the most important in their effects on accidents. Secondly, they feature prominently in the design standards; the principal differences between the LRRA and HLD standards lie in the design of bends. Lastly, alignment data were obtained from a different source from the geometric data discussed in Section 9.

The most common variables used to measure horizontal and vertical alignment were *bendiness* and *hilliness*:

- Bendiness is defined in COBA-9 (Department of Transport, 1981) as the absolute angle turned through along a link (counting left and right bends together) divided by the length of the link, and expressed in degrees per km.
- Hilliness is defined similarly as the total rise plus fall along a link, expressed in metres per km.

In many studies where details of bends and hills are required, it is common practice to measure them from maps, bends by measuring the angles between tangents to the line of the road and hills by counting contours. These measurements are, however, time-consuming, and cannot be done to any great accuracy, especially in the case of hills. Furthermore, more refined measures of alignment, such as the distribution of curvatures, cannot be made.

8.2 The High Speed Road Monitor

In this study, alignment data were obtained from measurements made by the High Speed Road Monitor (HRM). HRM is a device designed mainly for monitoring the surface texture of roads in order to indicate stretches of road that require maintenance. It measures 5 features of the road surface, namely the radius of curvature, gradient, cross-fall, macro-texture and rutting. Measurements are recorded every 10 metres, and each survey run covers up to 100 km of road. All trunk roads in England were surveyed during 1990 or 1991, and many have been re-surveyed since.

Curvature measurements are made by rotation counters on the wheels of the HRM, which is towed behind a van, and gradients by an inclinometer. The HRM therefore necessarily measures the curvature of the trailer path rather than that of the road itself, and cannot distinguish gradient from acceleration or trailer tilt.

8.3 Smoothing and bias correction

The techniques used for smoothing and bias correction are discussed here in relation to curvature data, though the same techniques were applied to the gradient data.

Smoothing was necessary because the individual 10-metre readings were highly variable and did not represent the true changes in the curvature of the road itself. The variability arose because of the unevenness of the road, the motion of the trailer, noise in the measuring devices and so on. If the data were not smoothed, an artificially high value for bendiness would be obtained because the noisy data represented many random changes in direction, albeit for a short (10 metres) distance. Smoothing was therefore applied to the data to remove high frequency components.

It was also essential to correct for bias. From the location of each 10-metre point and its curvature, it was possible to calculate the position of the next point and thus form a plot of the points as a simulated map. It was found that such maps tended to be in the form of a spiral because of a steady bias of a few degrees per km in all curvature readings. A linear correction was applied to all the curvature measurements for a given HRM run, adjusted so that the total angle turned through by the HRM trace on the simulated map along the length of the scheme was the same as the actual angle on an OS map.

8.4 Curvature and gradient variables

A range of variables relating to curvature and gradient were extracted for each minor link for use in the accident models. These are listed in Appendix 4.

The basic variables representing the average and range of curvatures on the minor link were the bendiness and the maximum curvature. Bendiness was calculated from the sum of the absolute angles turned through on each link. Curvature is defined as the reciprocal of the radius of curvature, and is measured in degrees per km. Thus, a radius of 1000 metres equates to a curvature of 57 degrees (1 radian) per km, a radius of 500 metres equates to 115 degrees (2 radians) per km, and so on. Maximum curvature implies minimum radius of curvature. The sum of curvatures was also extracted, from which the mean curvature could be calculated.

A set of variables representing the distribution of curvatures was also extracted, giving the proportion of each minor link having radii of curvature in each of several categories. The categories were chosen to correspond with the ranges labelled A, B, C and D in the Design Manual (Highways Agency, 1997a) for a design speed of 100 km/h. The important ranges from the point of view of accidents were those representing steeper curves (ranges C and D, and sharper curves constituting Departures from Standard). Range A represents slight bends permitting free overtaking on single-carriageway roads; range B represents slightly sharper bends where overtaking can take place on right hand downhill bends; range C represents the non-recommended intermediate range; and range D represents the range just above desirable minimum radius which is permitted for non-overtaking sections.

For gradients, a similar set of variables was extracted, namely the sum of absolute values (for hilliness), the maximum and minimum gradient, and the sums of gradients. The sum of gradients and sum of absolute gradients together gave the up- and down-hilliness, that is, the total rise or fall per km. The proportion of length with gradients in a number of categories (2 per cent steps up to 8 per cent, up and down) were also extracted.

9 Design variables

The term ‘design variables’ is used here to denote geometric features of the road design, such as carriageway width, verge width, and the presence of kerbs, hardstrips, etc, which might be expected to affect accident risk.

9.1 Use of videos

As part of the NIS (Network Information System) survey for DOT, video films were made of all trunk roads in England, and these were used, as in the dual-carriageway study, for observing the geometric variables of the schemes in this study. The main reason for preferring the use of videos to the more usual method of observing and measuring on site was cost. The use of videos avoided the need for a team of observers to assemble at widely scattered sites and traverse each scheme on foot, possibly more than once. Videos had obvious disadvantages, in that the observer was restricted to one view of the road, so examination of verges and side arms at junctions was limited. Also, measurements, for example of verge widths, were necessarily rather approximate. Videos had compensating advantages, in that it was easier to measure distances along the road (from video timings) and it was possible to retrace one’s steps and review a particular feature, or even go back to the video at a later date to check something.

The videos required for the study were identified from index maps held by DOT. 134 video tapes were required to cover 107 schemes of interest in both directions. It was found that 3 schemes were not covered by the video survey, so they were dropped from the study. VHS copies of the relevant tapes were made from the master tapes, which were loaned by HC Division in Leeds.

9.2 Extraction of data from videos

An initial pass through the tapes was made to locate the scheme and identify its links and junctions. This was followed by a further pass in which observations of the relevant variables were made at intervals of 5 seconds of video time (about 75 metres on the road).

A list of the variables measured in the geometric survey together with those derived from the basic variables is given in Appendix 4.

The video observations identified 429 minor links and included 12997 75-metre sections. There were 431 junctions of which 209 were 3-arm junctions, 199 were 4-arm junctions, 21 were 5-arm junctions, and 2 were 6-arm junctions.

10 Determination of scheme characteristics

10.1 Scheme age

In order to estimate the effect of scheme age on accident rate, the schemes were divided into 3 groups according to their year of opening: those opened in 1980 or earlier (the Old group), those opened between 1981 and 1985 (the Mid group), and those opened in 1986 or later (the New group). The age band for each scheme is shown as part of Appendix 5.

10.2 Carriageway width, kerbs and hardstrips

In earlier studies of accidents on single carriageways (see Section 3), carriageway width, the presence of a kerb and the presence of a hardstrip were found to be three of the most important variables in determining accident risk. These design features are also used in determining design speeds for new roads (see Section 10.3). The analysis of accident data in this study therefore concentrated on these design features. This section describes the methods used in assigning schemes to these categories and analyses the scheme structure accordingly.

10.2.1 Design standards

HLD design specifies that single-carriageway roads should be built with 1 metre hardstrips and without kerbs, except for lightly-trafficked roads with flows below about 8000 vehicles per day. Some modern roads, which were designed before HLD standards were introduced or where Departures from standard were authorised, may not conform to the standard in all respects.

The standard width for a 2-lane single carriageway is 7.3 metres (denoted by S2), but for heavily trafficked roads a wider standard width of 10 metres (denoted by WS2) can be used. There is no hard and fast traffic flow above which WS2 roads are recommended; the HLD standard suggests around 15000 vehicles per day, but taking note of local conditions which might make it uneconomic to provide for occasional heavy demand, or possible to provide WS2 for lighter traffic at little extra cost. The WS2 category also includes some older roads, built as 3-lane but later re-marked as 2 wide lanes.

10.2.2 Methods for determining width

In this study, carriageway width was calculated from the lane width and number of lanes. The lane width was estimated from videos at approximately 75-metre intervals along a road, using measurements on the screen, or visual clues such as passing vehicles. The average width for each link was calculated from these estimates, ignoring any stretches where the width was affected by junctions, climbing lanes etc. The link was then designated as wide (WS2, nominally 10m wide) or standard (S2, nominally 7.3m wide) according to whether the average width was greater than or less than 9m. In a few cases of very short links which were entirely affected by junctions, the width category of the adjacent link was used.

Altogether, there were 344 S2 links and 85 WS2 links. One scheme (scheme 63, A38 Saltash Bypass), which consisted of a single WS2 link, was entirely in tunnel, and was not used in studying the effect of width on accident rates.

In addition, each scheme as a whole was classified as S2 or WS2 according to its predominant width. This was done by averaging the widths of the constituent links, weighted by length, and checking whether the weighted average width was greater or less than 9m. 9 full schemes were designated as WS2. A further 6 schemes contained substantial lengths of predominantly WS2 carriageway; these hybrid schemes were split into WS2 and S2 sections and treated as two schemes for the purpose of this analysis. This gave a total of 15 WS2 schemes containing 52 links, and 93 S2 schemes containing 376 links.

The designation of schemes as S2, WS2 or hybrid is listed in Appendices 1 and 5.

10.2.3 Methods for determining kerbs and hardstrip widths

The presence of kerbs and the average width of hardstrip were calculated in a similar manner. The various edge treatment codes from the video data file were re-coded to 1 if a kerb was present and 0 if not. The 75-metre section kerb-codes and strip-widths were averaged over each minor link, ignoring any stretches affected by junctions, climbing lanes etc. An average, weighted by length, was then calculated for each scheme, and rounded to 0 or 1 to indicate the absence or presence of kerb or hardstrip over the majority of the scheme. The final codes for presence of kerb and hardstrip width for each scheme are shown in Appendix 5.

10.2.4 Analysis of scheme structures

A scattergram of carriageway widths against hardstrip widths was plotted for the 114 schemes (including split hybrids) in the study (Figure 10.1). The points were, of course, well scattered, because all measurements came from video estimation, and were subject to considerable measurement error. However, some features could be discerned.

There was a cluster of standard-width (S2) schemes with a carriageway width around 7.3m and a 1-metre hardstrip. A number of other schemes had a carriageway width of around 7.3m, but with varying hardstrip widths. The points with carriageway widths greater than 9m constituted the 15 WS2 schemes. Some were standard WS2 schemes with 1-metre hardstrips, but there were several with no hardstrip. There was also some indication that a number of schemes lay on the top-left to bottom-right diagonal. It seems likely that these points represented roads constructed with a standard 9.3m paved area (7.3m carriageway plus two hardstrips), and then re-marked with narrow, or no, hardstrips, to form a wider carriageway.

Schemes with kerbs are indicated by circles, and schemes without kerbs by triangles. As far as the S2 schemes are concerned, most of the schemes without hardstrips (bottom left) were kerbed, but many of those with hardstrips (top left) also had kerbs. However, the WS2 schemes separated almost completely into schemes with hardstrips but no kerbs (top right), and kerbed schemes without hardstrips (bottom right).

10.3 Speed variables

From the alignment and geometric data described above, a number of estimates of the speed of vehicles on each link were derived for testing in the models. The estimates were based on formulae used in cost benefit assessments

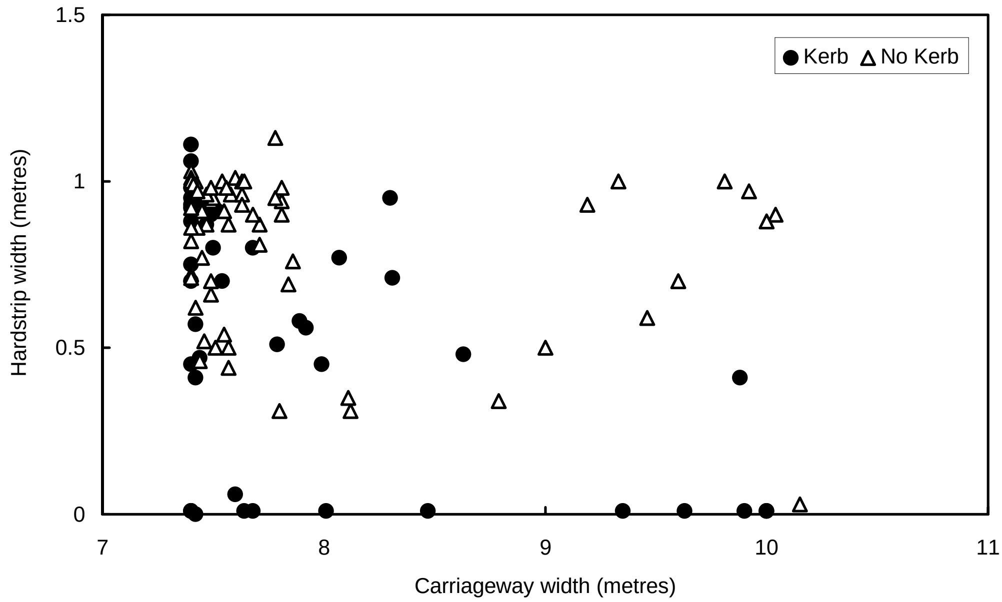

Figure 10.1 Relationship between carriageway width and hardstrip width

for new highways (Department of Transport, 1981, and Lee and Brocklebank, 1993).

While it seems intuitively obvious that, other things being equal, speed should have an effect on accident rates, it was not necessarily true that including speed in a model would improve the fit to the data, because speed is a dependent variable which is correlated with design features. Nevertheless, it seemed important to test this assumption by testing speed in the model.

In this study, there were no measurements of actual speeds on the road, because this would be a time-consuming process. Instead, speed estimates based on three formulae involving the geometric parameters were used as proxy speed variables.

Firstly, COBA-9 (Department of Transport, 1981) gave a formula, used in the HLD standard, for the mean free speed of light vehicles on single-carriageway roads as:

$$V = 76.5 - [0.0444*B] - [0.1111*H] - [0.0555*GRAD] - [0.01833*F] + [1.1*CWIDTH] - 10/(SWIDTH+VERGE+1) - [5*JNCS/CWIDTH] + [0.0167*VISI]$$

This formula was derived from data on vehicle speeds collected between 1977 and 1979 (Department of Transport, 1981).

Secondly, a later study (Lee and Brocklebank, 1993) derived a somewhat different relationship, which was incorporated in a new version COBA-10 (Highways Agency, 1997b), as follows:

$$V = 72.1 - [(0.09 - (0.075*NEW))*B] - [0.0007*(H*B)] - [0.11*GRAD] - [(0.015+(0.027*P))*F] + [2.0*CWIDTH] + [1.6*CONEDGE] + [1.1*SWIDTH] + [0.3*VERGE] - [1.9*JCNS] + [0.005*VISI].$$

In these formulae, the variables have the following meanings:

B	=	Bendiness
H	=	Hilliness
NEW	=	1 for roads of modern design, 0 otherwise
GRAD	=	Net gradient (zero for single carriageways when averaged over both directions)
F	=	vehicles / hour / direction
P	=	Proportion of HGVs
CWIDTH	=	Carriageway width
CONEDGE	=	1 or 0 depending on whether continuous edge lining is present
SWIDTH	=	Hardstrip width
VERGE	=	Verge width
JNCS	=	number of junctions or accesses
VISI	=	Harmonic mean visibility

The third speed variable used in this study was the Design Speed, which is defined in the HLD standard and is the most important variable to be considered in designing a new road. The design speed is derived by first calculating the mean free speed of light vehicles in the wet from a unified formula which combines the COBA-9 formulae for single and dual carriageways. The unified formula is:

$$V_{50\ wet} = 110 - L_C - A_C,$$

where A_C is an alignment constraint given by $A_C = 12 - VISI/60 + 2B/45$, and L_C is obtained from tables and depends on the carriageway width, the verge width and the number of accesses and junctions. For single carriageways the Layout Constraint varies between 17 and 33.

From the mean speed is calculated the 85th percentile speed ($V_{85\ wet}$), which is then allocated to one of a number of design speed ranges, designated 60B, 60A, 70B, 70A, 85B, 85A, 100B, 100A, 120B or 120A. Other aspects of road design can then be determined by reference to the design speed.

The number of schemes in the study which fell into each design speed group, and the average speeds within these groups according to the COBA-9 and COBA-10 formulae, are shown in Table 10.1. It should be noted that the Design Speeds were calculated from scheme characteristics, which can give a different calculated Design Speed from the speed used for designing the road in the first place. In particular, single-carriageway roads are designed for a maximum speed of 100 km/h, but the occurrence of straight and/or wide roads gave, in a few cases, a calculated Design Speed of 120 km/h.

Table 10.1 Average speeds according to the COBA-9 and COBA-10 formulae (km/h)

Design speed	*Number of schemes*	*COBA-9 average*	*COBA-10 average*	*Average difference*
85A	6	73.8	83.8	10.0
100B	34	79.0	85.5	6.5
100A	47	83.1	88.3	5.2
120B	16	85.6	91.6	6.0
All ranges	103	82.5	88.0	5.5

The average speed according to the COBA-10 formula was 88.0 km/h, with a range from 77.1 to 94.7 on individual minor links. The average speed according to the COBA-9 formula was 82.5 km/h, with a range from 66.1 to 88.3. The average difference was 5.5, with a range from 0.1 to 14.2; the COBA-10 formula gave a higher speed than the COBA-9 formula on all minor links. The scatter of speeds on individual links according to the two formulae, with design speeds indicated by different symbols, is shown in Figure 10.2.

The speeds according to the three methods of calculation (COBA-9, COBA-10 and design speed), plus $V_{50\ wet}$ and the difference between COBA-9 and COBA-10 speeds, were tested as variables in the regression models to see if speed was significantly correlated with accident rates. Even if the speed variable was statistically significant, the model could not distinguish whether higher speed caused more accidents, or whether the same design variables which result in higher speeds were the real cause.

11 Descriptive analysis of accidents

This section analyses the characteristics of the accidents on the 103 single-carriageway schemes in the study. The occurrence of missing values (for example, the day of week that the accident occurred not being recorded) means that the total number of accidents varies slightly from one table to another.

11.1 Number, frequency and severity of accidents

11.1.1 Average frequency and severity of accidents

Table 11.1 shows the numbers of accidents on each scheme (broken down into fatal, serious and slight injury accidents), together with the total scheme length and the number of years of accident data. From these data were derived, for each scheme, the average accident frequency (number of accidents per year), the accident density (number of accidents per km per year), and the severity ratio (the percentage of accidents that were fatal or serious). The total numbers of accidents, total scheme length and years of accident data are shown in Table 11.2.

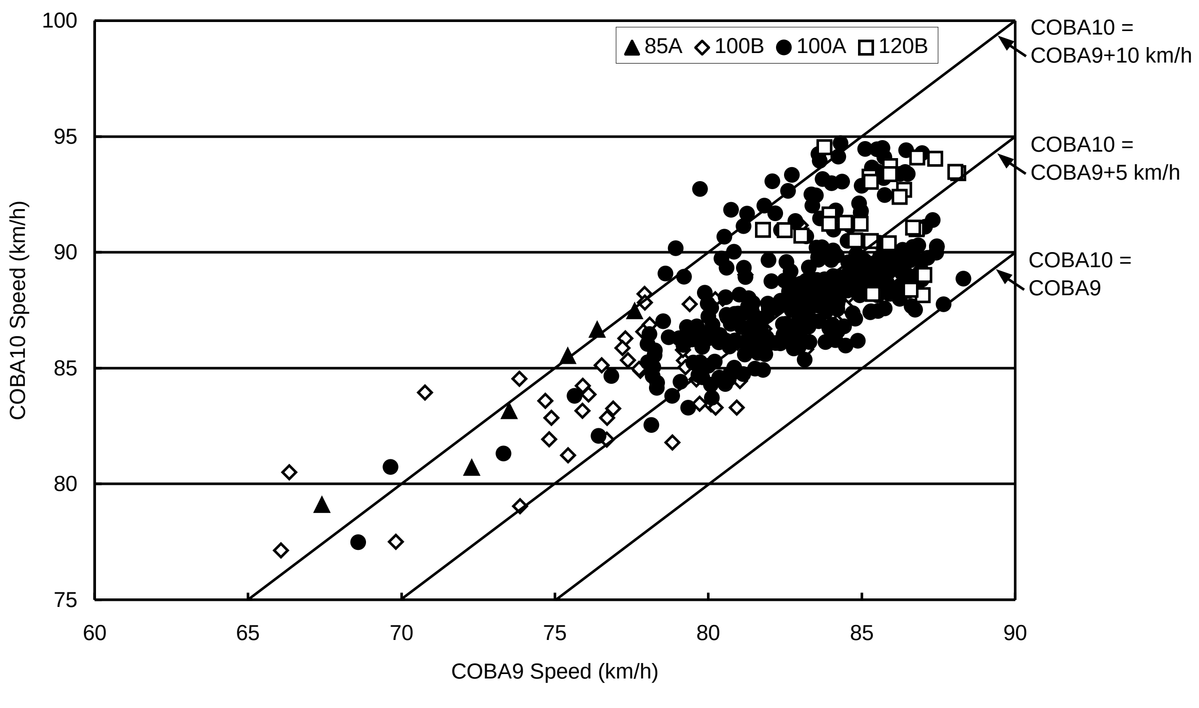

Figure 10.2 Relationship between COBA10 speeds and COBA9 speeds

Table 11.1 Number, frequency, and severity of accidents by scheme

Scheme	Scheme length (km)	Scheme years	Number of accidents: Fatal	Serious	Slight	Total	Accident frequency (acc/year)	Accident severity (% fatal & serious)	Accident density (acc/year per km of scheme)
001	3.71	4	1	7	8	16	4.00	50.0	1.08
002	1.52	8	1	6	27	34	4.25	20.6	2.80
003	1.51	9	0	1	3	4	0.44	25.0	0.29
004	5.13	2	2	4	7	13	6.50	46.2	1.27
005	4.74	5	3	11	22	36	7.20	38.9	1.52
006	6.05	6	7	19	14	40	6.67	65.0	1.10
007	4.56	6	2	6	14	22	3.67	36.4	0.80
008	12.42	4	2	12	35	49	12.25	28.6	0.99
009	4.02	3	0	3	5	8	2.67	37.5	0.66
010	10.36	8	7	35	47	89	11.13	47.2	1.07
011	7.59	8	3	19	26	48	6.00	45.8	0.79
012	1.92	4	1	13	33	47	11.75	29.8	6.12
013	1.96	8	1	3	10	14	1.75	28.6	0.89
014	16.40	8	5	38	43	86	10.75	50.0	0.66
015	6.95	5	5	12	13	30	6.00	56.7	0.86
017	0.69	12	0	6	9	15	1.25	40.0	1.81
018	3.62	4	3	4	16	23	5.75	30.4	1.59
019	3.00	8	1	4	13	18	2.25	27.8	0.75
020	3.67	3	2	4	7	13	4.33	46.2	1.18
021	4.68	1	1	1	5	7	7.00	28.6	1.50
022	3.07	8	0	5	2	7	0.88	71.4	0.29
023	7.90	1	0	0	1	1	1.00	0.0	0.13
024	5.15	2	2	0	13	15	7.50	13.3	1.46
025	3.99	5	0	5	11	16	3.20	31.3	0.80
027	6.13	5	4	7	21	32	6.40	34.4	1.04
028	3.06	11	0	12	18	30	2.73	40.0	0.89
029	3.12	6	0	3	8	11	1.83	27.3	0.59
030	2.78	4	0	4	1	5	1.25	80.0	0.45
031	2.85	4	0	3	17	20	5.00	15.0	1.75
032	1.69	4	0	3	11	14	3.50	21.4	2.07
033	4.57	4	0	3	24	27	6.75	11.1	1.48
034	2.14	7	1	1	2	4	0.57	50.0	0.27
035	8.52	5	1	10	28	39	7.80	28.2	0.92
036	5.72	7	2	3	9	14	2.00	35.7	0.35
037	8.31	7	3	13	53	69	9.86	23.2	1.19
038	4.32	9	3	7	14	24	2.67	41.7	0.62
039	1.10	12	0	2	1	3	0.25	66.7	0.23
040	2.95	6	3	8	15	26	4.33	42.3	1.47
041	9.59	5	4	15	20	39	7.80	48.7	0.81
043	7.50	12	3	26	30	59	4.92	49.2	0.66
044	3.64	4	1	3	15	19	4.75	21.1	1.30
045	4.97	3	0	2	15	17	5.67	11.8	1.14
046	2.74	6	2	4	15	21	3.50	28.6	1.28
047	1.13	12	0	3	14	17	1.42	17.6	1.25
048	4.04	12	2	17	28	47	3.92	40.4	0.97
049	2.84	8	1	4	6	11	1.38	45.5	0.48
050	2.45	3	0	3	7	10	3.33	30.0	1.36
051	9.67	12	4	16	40	60	5.00	33.3	0.52
052	1.50	8	0	1	4	5	0.63	20.0	0.42
053	5.27	12	2	6	16	24	2.00	33.3	0.38
054	16.81	12	11	63	90	164	13.67	45.1	0.81
055	10.04	12	10	20	34	64	5.33	46.9	0.53
056	12.7	12	10	50	74	134	11.17	44.8	0.88
057	2.1	1	0	0	2	2	2.00	0.0	0.95
058	2.76	8	0	4	10	14	1.75	28.6	0.63
059	1.23	12	2	5	8	15	1.25	46.7	1.02
060	3.04	12	10	13	17	40	3.33	57.5	1.10
061	2.96	12	3	18	37	58	4.83	36.2	1.63
062	3.02	8	3	10	18	31	3.88	41.9	1.28
063	0.80	2	0	2	12	14	7.00	14.3	8.75
064	6.21	5	2	2	7	11	2.20	36.4	0.35

Table 11.1 Number, frequency, and severity of accidents by scheme (Continued)

Scheme	Scheme length (km)	Scheme years	Number of accidents: Fatal	Serious	Slight	Total	Accident frequency (acc/year)	Accident severity (% fatal & serious)	Accident density (acc/year per km of scheme)
065	4.30	12	1	4	8	13	1.08	38.5	0.25
066	9.86	6	4	8	21	33	5.50	36.4	0.56
067	12.76	1	0	2	1	3	3.00	66.7	0.24
068	7.77	3	4	1	9	14	4.67	35.7	0.60
069	25.07	2	3	4	13	20	10.00	35.0	0.40
070	18.85	1	0	3	8	11	11.00	27.3	0.58
071	2.94	3	0	7	5	12	4.00	58.3	1.36
072	5.38	8	0	11	18	29	3.63	37.9	0.67
073	1.43	7	0	1	2	3	0.43	33.3	0.30
074	2.94	1	0	2	3	5	5.00	40.0	1.70
075	5.83	1	0	1	2	3	3.00	33.3	0.51
076	2.01	7	2	6	8	16	2.29	50.0	1.14
077	6.47	6	2	14	38	54	9.00	29.6	1.39
080	0.86	12	0	16	17	33	2.75	48.5	3.20
081	13.38	2	7	4	13	24	12.00	45.8	0.90
082	0.56	1	0	0	1	1	1.00	0.0	1.79
084	1.69	4	0	1	1	2	0.50	50.0	0.30
085	2.25	2	1	0	1	2	1.00	50.0	0.44
086	9.66	2	1	2	8	11	5.50	27.3	0.57
087	3.06	12	1	10	14	25	2.08	44.0	0.68
089	6.08	3	0	1	4	5	1.67	20.0	0.27
090	2.16	1	0	1	3	4	4.00	25.0	1.85
091	3.39	7	0	5	10	15	2.14	33.3	0.63
092	6.08	2	0	1	3	4	2.00	25.0	0.33
093	7.71	6	3	2	7	12	2.00	41.7	0.26
094	1.76	7	0	2	7	9	1.29	22.2	0.73
095	8.14	11	1	8	22	31	2.82	29.0	0.35
096	1.38	2	1	0	0	1	0.50	100.0	0.36
097	6.37	4	0	9	15	24	6.00	37.5	0.94
098	10.62	4	2	11	26	39	9.75	33.3	0.92
100	4.56	10	0	2	10	12	1.20	16.7	0.26
101	2.91	10	1	4	13	18	1.80	27.8	0.62
102	7.10	3	0	8	12	20	6.67	40.0	0.94
103	5.81	7	3	18	29	50	7.14	42.0	1.23
104	3.91	7	2	9	11	22	3.14	50.0	0.80
105	3.96	2	1	6	4	11	5.50	63.6	1.39
106	5.54	2	0	2	9	11	5.50	18.2	0.99
107	0.95	6	0	2	2	4	0.67	50.0	0.70
108	2.79	6	2	2	13	17	2.83	23.5	1.02
109	11.20	2	5	9	9	23	11.5	60.9	1.03
110	2.12	6	0	2	9	11	1.83	18.2	0.86
All schemes	540.49	617	183	785	1534	2502	4.06	38.7	0.81

Table 11.2 Number, frequency, and severity of accidents - Summary

Summary variables		*Value*
Total length of schemes		540.5 km
Total scheme-years		617
Total number of accidents		2502
of which:	Fatal	183
	Serious	785
	Slight	1534
Average Accident Frequency		4.1 acc/year/scheme
Average Accident Density		0.81 acc/year per km of scheme
Average Accident Severity		38.7% fatal and serious

The mean accident severity ratio was 39 per cent. Neglecting those schemes with fewer than 10 accidents in total (which led to large uncertainties in the severity ratio), the distribution of severities over the individual schemes had a minimum and maximum of 11 and 65 per cent respectively. The mean accident severity was high compared with the national average for non-built up ‘A’ roads (28.6 per cent). The average fatality ratio (ratio of fatal accidents to all accidents) for the study schemes was 7.3 per cent, which was also high compared to the national fatality ratio (4.3 per cent). This is probably because the study schemes were trunk roads carrying more and faster traffic than A-roads generally.

The average accident frequency for the schemes studied was 4.06 accidents per year, with a minimum of 0.25 on scheme 39 (A1 Warenford Diversion) and a maximum of 13.67 on scheme 54 (A66 Chapel Brow to Peel Wyke Improvement). This latter scheme was the second-longest (16.8 km), with a high number of accidents (164) over 12 scheme-years, while the former was one of the shortest (1.1 km). Since accident frequency takes no account of scheme length, it is likely to be higher on longer schemes.

The average accident density (accidents per km per year) was 0.81 accidents per year per km. Two schemes had unusually high accident densities, but they were relatively short, so might have been affected by nearby junctions. They were also non-typical in that one (scheme 12: A47 Great Yarmouth Western Bypass) was very close to the town and was almost urban in character, and the other (scheme 63: A38 Saltash Bypass) was a section of tunnel with sharp curves and 3-lane tidal flow. Excluding these two schemes, the maximum and minimum accident densities were 2.07 and 0.13 respectively.

11.1.2 Number, frequency and severity of accidents by location

The distribution of accidents by location (link or junction) is shown in Table 11.3.

The accident frequencies for link and for junction accidents were similar, at 2.10 accidents per year for link accidents and 1.96 for junction accidents, the latter comprising 0.63 for major junctions and 1.32 for minor junctions. These figures were scheme-based statistics and therefore varied according to the length of the scheme and the number of junctions.

The accident severity for link accidents was 44 per cent, and for junction accidents 33 per cent (23 per cent for major junctions and 38 per cent for minor junctions). The main reason for accident severity being higher for link accidents was a much higher fatality ratio (11 per cent compared to 3 per cent).

11.1.3 Number, frequency and severity of accidents by junction type

Table 11.4 compares the accident frequency and severity between junction types. Accident frequencies were significantly higher for roundabouts (0.65 per junction-year) and crossroads (0.55) than the overall average of 0.42, and lower at T-junctions (0.28). However, the severity at roundabouts (22 per cent) was lower than average for junctions (33 per cent), and was high (41 per cent) at T-junctions. This indicated that a greater number of accidents at roundabouts were slight-injury accidents. The frequencies for roundabouts are close to those for all major junctions (section 11.1.2) because only one major junction was not a roundabout.

11.1.4 Number, frequency and severity of accidents by accident group

Table 11.5 presents the number of accidents classified as fatal, serious or slight, broken down by accident groups. The 25 accident groups refer to different vehicle manoeuvres and are listed in Appendix 3. Groups A to I are junction accidents and groups J to Z link accidents.

From Table 11.5, accident group X, which represents accidents involving head-on collisions without overtaking, had the highest total number of accidents (11.2 per cent of the total) and the highest number of fatal accidents (26 per cent of fatal accidents). Among the junction accident groups, groups B (1-vehicle accidents) and group E (2-vehicle accidents involving a right turn into the major road) had the highest numbers, with 10 per cent of all accidents each. The accident group with the lowest number of accidents (0.1 per cent of the total), all of which were slight injury accidents, was accident group P (single-vehicle accidents on a right-hand bend).

Ignoring groups with fewer than 20 accidents, the group with the highest severity was group U (head-on collisions involving overtaking), where 71 per cent of accidents were fatal or serious. The lowest severity was in group H (junction accidents involving two vehicles travelling in the same direction), with 11 per cent.

Table 11.3 Number, frequency, and severity of accidents by location

Location	Scheme years	Number of accidents: Fatal	Serious	Slight	Total	Accident frequency (acc/year/ scheme)	Accident severity (% fatal & serious)	Accident density (acc/year per km of scheme)
Major junction	617	3	85	303	391	0.6	22.5	0.13
Minor junction	617	38	273	505	816	1.3	38.1	0.26
Link	617	142	427	726	1295	2.1	43.9	0.42
Total	617	183	785	1534	2502	4.1	38.7	0.81

Table 11.4 Number, frequency, and severity of accidents by junction type

Junction type	*Number of junctions*	*Scheme years*	*Number of accidents*				*Accident frequency (acc/year)*	*Accident severity (% fatal & serious)*
			Fatal	*Serious*	*Slight*	*Total*		
Scheme end	25	173	0	8	32	40	0.23	20.0
Roundabout	101	534	4	73	274	351	0.66	21.9
Traffic signals	1	6	0	0	1	1	0.17	0.0
Slip road	7	44	1	6	17	24	0.55	29.2
T-junction	189	1251	19	125	205	349	0.28	41.3
Staggered junction	62	389	13	59	121	193	0.50	37.3
Crossroads	58	452	4	86	157	247	0.55	36.4
Mini-roundabout	1	12	0	5	5	7	0.58	71.4
Left in/left out junction	2	8	1	1	1	3	0.38	66.7
Other type of junction	6	35	0	1	3	4	0.11	25.0
All junction accidents	452	2904	42	364	813	1219	0.42	33.3

Table 11.5 Number, frequency, and severity of accidents by accident group

Accident Group (see Appendix 3)	*Number of accidents*				*Accident frequency (acc/year /scheme)*	*Accident severity (% fatal & serious)*	*Accident density (acc/year / km)*	*% of all accidents*
	Fatal	*Serious*	*Slight*	*Total*				
Junction accidents:								
A	2	9	7	18	0.03	61.1	0.006	0.7
B	3	75	179	257	0.42	30.4	0.083	10.3
C	0	27	63	90	0.15	30.0	0.029	3.6
D	9	62	95	166	0.27	42.8	0.054	6.6
E	14	80	156	250	0.41	37.6	0.081	10.0
F	3	37	65	105	0.17	38.1	0.034	4.2
G	0	3	3	6	0.01	50.0	0.002	0.2
H	2	11	109	122	0.20	10.7	0.040	4.9
I	8	54	131	193	0.31	32.1	0.063	7.7
Link accidents:								
J	17	20	23	60	0.10	61.7	0.019	2.4
K	3	15	33	51	0.08	35.3	0.017	2.0
L	0	1	11	12	0.02	8.3	0.004	0.5
M	0	4	13	17	0.03	23.5	0.006	0.7
N	6	44	84	134	0.22	37.3	0.043	5.4
O	2	8	6	16	0.03	62.5	0.005	0.6
P	0	0	3	3	0.01	0.0	0.001	0.1
Q	4	22	54	80	0.13	32.5	0.026	3.2
R	1	21	37	59	0.10	37.3	0.019	2.4
S	3	12	22	37	0.06	40.5	0.012	1.5
T	4	26	53	83	0.14	36.1	0.027	3.3
U	29	67	40	136	0.22	70.6	0.044	5.4
V	15	43	70	128	0.21	45.3	0.042	5.1
W	8	37	137	182	0.30	24.7	0.059	7.3
X	48	100	131	279	0.45	53.0	0.090	11.2
Z	2	7	9	18	0.03	50.0	0.006	0.7
All accidents	183	785	1534	2502	4.06	38.7	0.811	100.0

11.2 Breakdowns of accidents

11.2.1 Accidents by number of casualties

Table 11.6 shows the distribution of accidents, classified as link or junction accidents, by the number of casualties involved. A high proportion of accidents resulted in one casualty, with percentages of 55 per cent on links, 72 per cent at major junctions, and 51 per cent at minor junctions. Three link accidents in this study resulted in 20 or more casualties, all involving buses. Two of these involved fatalities.

The average number of casualties per accident was 1.94 for link accidents, 1.86 for accidents at minor junctions, and 1.42 at major junctions. The number of fatal casualties was much higher in link accidents (0.13) than at junctions (0.05 at minor junctions and 0.01 at major junctions).

Table 11.6 Accidents by number of casualties

Type	*Number of accidents*	*% by number of casualties*						*Average casualties/accident*			
		1	*2*	*3*	*4*	*5*	*6+*	*Fatal*	*Serious*	*Slight*	*Total*
Major junction	391	72	20	6	1	1	0	0.01	0.26	1.14	1.42
Minor junction	816	51	29	12	5	2	1	0.05	0.49	1.30	1.86
Link	1295	55	24	9	6	3	3	0.13	0.58	1.18	1.94
Total	2502	56	25	10	5	2	2	0.09	0.50	1.21	1.83

11.2.2 Accidents by number of vehicles and accidents involving pedestrians

Table 11.7 analyses the accidents by the numbers of vehicles involved and the involvement of pedestrians. The majority of accidents (58 per cent) involved 2 vehicles. A higher proportion of junction accidents than link accidents involved 2 vehicles (57 per cent at major junctions and 76 per cent at minor junctions, compared to 47 per cent on links), whereas more link accidents involved 3 or more vehicles (23 per cent).

Table 11.7 Accidents by number of vehicles and accidents involving pedestrians

Type	*Number of accidents*	*Percent by number of vehicles*						*Accidents involving pedestrians (Percent of all accidents)*
		1	*2*	*3*	*4*	*5*	*6+*	
Major junction	391	37	57	5	1	0	0	1.0
Minor junction	816	12	76	9	2	1	0	1.7
Link	1295	30	47	17	4	1	1	4.6
Total	2502	26	58	13	3	1	0	3.1

The percentage of accidents involving pedestrians was comparatively small (3.1 per cent), with a higher proportion for link accidents (4.6 per cent) than junction accidents. This can be compared with accidents nationally where pedestrians constitute 17 per cent of casualties. This is explained by the rural nature of the roads in the study.

11.2.3 Vehicle involvements by type of vehicle and numbers of pedestrians involved

Table 11.8 shows the number of vehicles involved in accidents, by type of vehicle, and the number of pedestrians involved. A single accident contributes one or more vehicle involvements, and vehicle-pedestrian accidents contribute a pedestrian involvement and one or more vehicle involvements.

As expected, most accidents involved cars or taxis; 71 per cent of the 3558 vehicle involvements. About half of the car/taxi and pedal cycle involvements occurred in link accidents, whereas well over half the motor cycle involvements occurred at junctions. Both buses/coaches and HGVs had twice as many involvements in link accidents as junction accidents.

11.3 Breakdown of accidents by time period

11.3.1 Variation of accident numbers by year

The numbers of accidents by year (1979-1990) are shown in Table 11.9, broken down into link, major junction and minor junction accidents. Over 70 per cent of the accidents occurred in the later years (1986 to 1990), because more schemes were open, and because of traffic growth.

In order to estimate the actual trend in accidents over time, a constant set of schemes was examined, namely those which opened before 1979 which therefore provided accident data throughout the period 1979-90. The growth rate in accidents on these schemes (calculated by fitting a regression line to the log of the accident numbers) was 3.3 per cent per year for all accidents, and 3.2 per cent per year for link accidents alone. However, the confidence intervals on these growth rates were large, ± 2.2 per cent per year for total accidents and ± 3.0 per cent per year for link accidents. This implied that the growth rate for total accidents was significantly different from zero (at the 5 per cent level), but the rate for link accidents was not.

The growth in traffic over the same period was 4.5 per cent per year (Section 7.2). This implied a decrease in the total accident rate of 1.2 per cent per year and a decrease in the link accident rate of 1.3 per cent per year. Neither of these values were statistically significantly different from no trend.

Table 11.8 Vehicle involvements by type of vehicle, and pedestrian involvements

Type	*Vehicle type*						*Total vehicles involved*	*Total Pedestrians involved*
	Pedal cycle	*Motor cycle*	*Car/Taxi*	*Bus/Coach*	*LGV*	*HGV*		
Major junction	28	96	468	3	25	53	673	4
Minor junction	41	122	1260	12	106	109	1650	14
Link	62	163	1830	30	184	313	2582	62
Total	131	381	3558	45	315	475	4905	80

Table 11.9 Accidents by year

	Year												
Type	*1979*	*1980*	*1981*	*1982*	*1983*	*1984*	*1985*	*1986*	*1987*	*1988*	*1989*	*1990*	*Total*
All schemes													
Major junction	6	9	1	3	11	15	21	40	56	61	73	95	391
Minor junction	21	25	30	31	35	58	60	86	81	90	148	150	815
Link	27	50	40	40	68	59	79	137	132	181	232	251	1296
Total	54	84	71	74	114	132	160	263	269	332	453	496	2502
Schemes open before 1979													
Major junction	6	8	1	2	2	3	0	0	0	6	5	5	38
Minor junction	21	24	26	24	21	29	30	26	26	33	30	41	331
Link	27	42	37	32	39	27	32	42	36	36	43	57	450
Total	54	74	64	58	62	59	62	68	62	75	78	103	819
Schemes open before 1986													
Major junction	6	9	1	3	11	15	21	36	33	35	49	51	270
Minor junction	21	25	30	31	35	58	60	80	59	64	88	93	644
Link	27	50	40	40	67	59	79	123	105	121	143	164	1018
Total	54	84	71	74	113	132	160	239	197	220	280	308	1932

A similar exercise was carried out using the set of schemes which were open throughout the period 1986-1990. This gave more accidents, but fewer years, to work with. The growth rates were larger, 5.9 per cent per year for both total and link accidents, but the confidence intervals were also considerably larger at ± 11 per cent per year. As a result, neither growth rate was significantly different, either from zero, or from the traffic growth rate.

The conclusion was that there was no significant trend in either accident numbers or rates.

11.3.2 Variation of accident numbers by month of year

Table 11.10 presents the distribution of accidents by month. August had the highest percentage of accidents (10.4 per cent), whereas February had the lowest (6.3 per cent). The corresponding accident frequencies were 5.1 and 3.1 accidents per scheme per year (this can be interpreted as the accident frequency for a year of Augusts or Februaries respectively).

Table 11.10 Accidents by month of year

Month	*Percent of accidents*	*Accident frequency (accs per year)*	*Accident ratio[1] Study schemes*	*Casualty ratio National statistics[2]*
January	7.0	3.40	0.84	0.87
February	6.3	3.05	0.75	0.81
March	7.1	3.44	0.85	0.93
April	7.4	3.58	0.88	0.90
May	8.0	3.89	0.96	1.01
June	8.3	4.05	1.00	1.01
July	8.0	3.91	0.96	1.07
August	10.4	5.08	1.25	1.07
September	9.6	4.69	1.16	1.02
October	9.2	4.47	1.10	1.14
November	9.4	4.57	1.13	1.14
December	9.3	4.55	1.12	1.04

[1]*Accident ratio* $= \dfrac{\textit{Number of accidents for specific month}}{\textit{Average number of accidents for all months}}$

[2]*Casualty ratio is defined similarly to accident ratio, and is based on published casualty data (Department of Transport 1991b, Table 27) relating to all roads, as relevant accident data were not available.*

Table 11.10 also gives accident ratios, that is, the ratio of accident frequency in the month concerned compared to the annual average. These can be compared with national statistics (Department of Transport, 1989b), though since the appropriate accident data were not readily available, the figures given are the casualty ratios, that is, the casualty frequency by month compared to the annual average. The national statistics refer to rural and urban roads.

11.3.3 Variation of accident numbers by day of week

Table 11.11 shows the distribution of accidents by day of week. Saturday had the highest percentage (17.3 per cent), closely followed by Sunday (17.0 per cent), whereas Wednesday had the lowest (11.6 per cent). The corresponding accident frequencies were 4.9, 4.8 and 3.3 accidents per year.

Like Table 11.10, Table 11.11 gives accident ratios, which can be compared with national statistics (Department of Transport, 1989b) for driver involvements.

12 Tabular analysis of accident rates

12.1 Introduction

This section presents a tabular analysis of overall accident rates, in which the effect on accident rates of carriageway width and the presence of kerbs and hardstrips were explored. In addition, the variation of accident rates with the opening date, or age, of the scheme was investigated.

Tabular analysis of accident rates was used in order to provide a preliminary overview of the main variables

Table 11.11 Accidents by day of week

Day of week	Percent of accidents	Accident frequency (accs per year)	Accident ratio(1) Schemes in study	Driver involvement ratio National Statistics(2)
Monday	13.94	3.96	0.98	0.97
Tuesday	13.42	3.81	0.94	0.96
Wednesday	11.55	3.28	0.81	0.98
Thursday	13.38	3.80	0.94	1.05
Friday	13.30	3.78	0.93	1.21
Saturday	17.34	4.92	1.21	1.03
Sunday	16.98	4.82	1.19	0.80
Not stated	0.08			

[1]Accident ratio = Number of accidents for specific day of week / Average number of accidents for all days

[2]Driver involvement ratio is defined similarly to accident ratio, and is based on published data (Department of Transport 1991b, Table 35) on the numbers of motor vehicle drivers involved in accidents on all roads, as relevant accident data were not available.

which affect accidents, and to give a lead as to which variables should be included in the regression modelling. The tabular form has the advantage of being easier to digest than regression model results, since accident rates are intuitively meaningful in a way that regression coefficients are not. This is also a disadvantage, because accident rates assume a simple linear proportional relationship between accidents and traffic, which is only approximately correct.

12.2 Methodology

The methods used for calculating carriageway width, hardstrip width and presence of kerbs, and of assigning them to the categories wide/standard width, presence/ absence of kerb and presence/absence of hardstrip were discussed in Section 10.2. The identification of 6 schemes as hybrid schemes, with portions of wide and standard width carriageway, and the way in which these were divided into part schemes for analysis, was also discussed.

In this part of the analysis, accident data from only the last 5 available years (1986-1990) were used. This was to minimise the effect of any underlying trend in accident numbers which might occur even when there was no change in traffic flow or layout. This approach also permitted an analysis of the effect of scheme age on accident rates. While the age of a scheme as such should not affect accident rates, it can act as a proxy for design changes or for location (if newer schemes were predominately bypasses, or in hilly terrain, for example).

The use of just 5 years of data also minimised the effect of traffic growth in the analysis. This was, however, allowed for by using a weighted average traffic level in calculating the accident rates. This average was calculated from the 1990 traffic assuming an exponential growth factor of 4.5 per cent per year and allowing for sites which opened during the 5 year period and for occasional missing years of accident data.

Three types of accident rates were calculated:

i Link-only accident rates, that is, taking account of only those accidents classified as link accidents. This form of analysis should give results comparable with the regression models.

ii Major link accident rates, that is, including all accidents which occur on a major link, including those at minor junctions. This form of analysis gives rates which are comparable with those in earlier unpublished work by DOT.

iii Scheme accident rates, that is, including all link, minor junction and major junction accidents. This form of analysis is comparable with the rates used in COBA-9 for estimating accidents on a network of roads.

In all cases, accident rates were calculated as scheme averages. In order to avoid double-counting of junction accidents, which would bias the accident rates upwards, the number of accidents at all junctions at the ends of schemes was halved, the rationale being that the other half of the accidents belonged to the adjoining stretch of road. Similarly when a hybrid scheme was split, the accidents at the common junction were divided equally between the two part-schemes. In addition, accidents were assumed to belong to the major road at a junction, so in those few cases where the scheme road encountered a more important road, accidents at that junction were not counted. A ‘more important’ road was defined as a motorway, or a trunk A-road with a lower road number than the scheme road.

It should be noted that, in considering major link accident rates, the rates obtained depended on the number of minor junctions occurring on each major link. Likewise, scheme rates depended on the number of major and minor junctions. When comparing these accident rates between schemes, there is an implicit assumption that junctions occur with a similar frequency on all schemes. If a scheme had more junctions than average it would be likely to have a high scheme and major-link accident rate.

12.3 Results: overall accident rates

Table 12.1 shows a summary of the averages and ranges of accident rates, and Appendix 5 shows the full analysis of accident rates for each scheme. The overall average accident rates on the schemes studied were:

Link-only rate:

(excluding all junction accidents)	0.125 accidents per million vehicle-km

Major-link rate:

(including accidents at minor junctions but not major junctions)	0.197 accidents per million vehicle-km

Scheme rate:

(including accidents at both major and minor junctions, with adjustments for scheme ends and junctions with more important roads as previously described)	0.231 accidents per million vehicle-km

Table 12.1 Averages and ranges for scheme accident rate (1986-1990)

	Accident rates per 100MVehkm			*Junction rates per 100MVeh*	
	Link	*Major link*	*Scheme*	*Major junction*	*Minor junction*
Average	12.5	19.7	23.1	21.7	13.9
Median	11.1	19.1	22.7	16.5	10.5
Lower quartile	7.3	10.8	14.3	6.8	3.5
Upper quartile	18.4	26.0	28.9	28.1	17.4
Minimum (excluding schemes with small numbers of accidents)	2.0	2.0	2.4	3.4	1.0
Maximum (excluding schemes with small numbers of accidents)	45.6	68.7	68.7	88.1	51.1

a) Acc Yrs = number of years for which accident data are available
b) Total vehicle-km is used for calculation of link accident rates
c) Total vehicles throughput = AADT summed over Acc Yrs
d) Number of major and minor junctions on the scheme is adjusted for scheme ends and junctions with more important roads, see text
e) Major and minor junction vehicle throughput = the number of junctions times the total vehicle throughput, for calculation of junction accident rates
f) Accident rates for links only, major links and schemes per 100 million vehicle-km.
g) Accident rates for major and minor junctions per 100 million vehicles passing through the junctions.
h) These results are based on the following data:

Number of schemes		*108*
Number of Major junctions		*84.5*
Number of Minor junctions		*290.5*
Total length (km)		*531.3*
Total accident years		*425.0*
Total traffic (MVehkm)		*7090.1*
Number of accidents:	*Link*	*889.0*
	Major junction	*239.5*
	Minor junction	*511.0*

The link-only and the scheme rates can be compared with the rates described in COBA-9 (November 1993 revision) as Link-only and Combined Low rates (applicable to new HLD-like roads). The rates derived here were a little lower than the COBA-9 rates, which were 0.17 and 0.25 per MVkm, respectively. The COBA-9 rates for existing rural A-roads, 0.22 and 0.33 per MVkm, were much higher.

Work by DOT derived accident rates of 0.16 to 0.22 per MVkm for 7.3m roads of modern design, the former figure being derived from data for 1981-84 and the latter from 1987-90. The rates for 10m roads were a little lower. These figures compare well with the Major link rate above, 0.197, which contained a few 10m samples.

The spread of accident rates among the schemes studied was large, ranging from a minimum of zero (because scheme 163 did not have any accidents at all during the period considered) to a maximum of 0.456 per MVkm for link-only rates (compared with the mean of 0.125). The maximum could be strongly affected by one scheme with a high rate (which might result from quite a small number of accidents). A better measure of the top of the range is the 90th percentile, which was 2.0 times the median for link-only rates, 1.8 times it for major link and scheme rates, and about 3 times it for junction rates.

12.4 Results: breakdown of accident rates

Table 12.2 presents a breakdown of accident rates by the main characteristics of scheme age, carriageway width, and presence of kerbs and hardstrips. The significance level of the difference between the categories in each breakdown was tested with a χ^2 test and indicated in the table. The overall link accident rate (row 1) was 0.125 per million vehicle-km.

12.4.1 Analysis by age

Table 12.2 shows that for link-only rates, for scheme rates, and for minor junction rates, there was no significant difference between the schemes according to their age - in other words, older schemes did not have a significantly higher or lower accident risk than new ones. There was, however, a very highly significant difference in major link rates and major junction rates according to age. The former could be attributed to the fact that while the total number of junctions was about the same (0.7 per km) throughout (which explains why there were no significant differences in scheme rates), schemes in the Old age band (pre-1980) had significantly fewer major junctions, 0.06 per km compared to 0.20 per km in the New band (1986-90), with correspondingly more minor junctions. The major link rates on older roads would thus include more minor junction accidents.

12.4.2 Analysis by carriageway width

All three link accident rates were significantly lower for wide (WS2) carriageways than for standard (S2) widths, the difference for major link and scheme rates being very highly significant. The major junction rate, but not the minor junction rate, was also significantly lower for WS2.

Again, part of the explanation was that WS2 roads had significantly more major junctions and fewer minor junctions than S2s, though this could be a function of age (WS2 roads tended to be newer) rather than of design. It is therefore advisable to take more notice of the difference in link-only rates rather than those which include junctions.

The link-only rate was significantly lower for WS2 roads, but only at the 5 per cent level. An obvious interpretation of this result is that WS2 roads are safer

Table 12.2 Breakdown of accident rates by scheme characteristics (1986-90)

		Accident rates per MVkm			*Junction rates per MVeh*	
		Link-only	*Major link*	*Scheme*	*Major junction*	*Minor junction*
Overall rate		0.125	0.197	0.231	0.217	0.139
By age	Old	0.135	0.238	0.249	0.169	0.142
	Mid	0.128	0.181	0.228	0.281	0.137
	New	0.113	0.180	0.218	0.163	0.133
	Significance level	*NS*	***	*NS*	***	*NS*
By carriageway width	S2	0.131	0.207	0.239	0.237	0.138
	WS2	0.095	0.144	0.189	0.160	0.135
	Significance level	*	***	***	**	*NS*
By hardstrip	No strip	0.143	0.228	0.265	0.185	0.136
	Strip	0.120	0.189	0.222	0.229	0.138
	Significance level	*	***	***	*NS*	*NS*
By kerb	No kerb	0.119	0.188	0.222	0.221	0.143
	Kerb	0.136	0.215	0.248	0.210	0.131
	Significance level	*NS*	*NS*	*	*NS*	*NS*
By kerb/strip combinations	No kerb,No strip	0.127	0.211	0.250	Not tested	Not tested
	Kerb, No strip	0.155	0.239	0.275		
	No kerb, strip	0.118	0.184	0.218		
	Kerb, strip	0.126	0.200	0.232		
	Significance level	*	**	*		
By kerb/strip combinations on S2 only	No kerb,No strip	0.128	0.195	0.238	Not tested	Not tested
	Kerb, No strip	0.181	0.280	0.313		
	No kerb, strip	0.124	0.198	0.228		
	Kerb, strip	0.126	0.200	0.232		
	Significance level	**	***	***		

Significance levels:

	*	*Significant at the 5 per cent level*
	**	*Significant at the 1 per cent level*
	***	*Significant at the 0.1 per cent level*
	NS	*Not significant at the 5 per cent level*

than S2 roads. However, in the regression carriageway width only emerged as a significant variable when averaged over schemes, and not over links. The lower accident rate in this simpler analysis was also partly due to correlation with traffic levels and scheme age.

12.4.3 Analysis by presence of hardstrip and kerb

From the table, it can be seen that the presence of a 1-metre hardstrip reduced the link-only accident rate from 0.143 to 0.120, a change which was significant at the 5 per cent level. The changes in the major link and the scheme rates were very highly significant. The presence of a kerb, however, was not significant, except at the scheme level.

An analysis of all four kerb and hardstrip configurations revealed a significant effect, in that the accident rate for roads with a kerb but no hardstrip was considerably higher than for other roads. This effect was especially marked when the analysis was confined to S2 roads, where the rate for roads with kerb and no hardstrip was 50 per cent higher than average (link-only rate 0.181 compared with the overall average of 0.125). There were no comparable effects on WS2 roads, but the sample of these was too small to allow such a detailed breakdown.

12.5 Conclusions from the tabular analysis

The conclusion from this section is that wide single carriageways (WS2) appeared to have a lower accident rate than standard-width (S2) roads. This was, however, partly due to the fact that WS2 roads on average had higher traffic flows. There is considerable evidence from other studies that accident frequency depends on traffic flow in a non-linear relationship, leading to lower accident rates at higher traffic flows. This would imply a lower accident rate on WS2 roads. Accident rates found in this study were similar to earlier studies in COBA-9 (Department of Transport 1981).

The presence of a 1-metre hardstrip gave a significant reduction in link accident rates. There was an indication that the accident rate was higher on S2 width roads with kerbs but no hardstrips, but this was not proven.

13 Time trend analysis

13.1 The significance of accident trends

It is well known that the overall accident rate on British roads has been falling steadily since the 1950s at least. For example, in 1981 the accident rate for all roads was

0.90 accidents per million vehicle-km (Department of Transport, 1992), while by 1991 it had dropped to 0.57, a fall of 36 per cent. This continuing fall is believed to be due to improvements in design standards for roads and vehicle design, to improvements in driver behaviour through education campaigns and legislation, and to a change in the composition of traffic (for example, fewer two-wheelers). In addition, an increase in average vehicle flows leads to a lower accident rate because accident frequency does not depend linearly on traffic flow.

On a given link, whose length and design features are constant, the trend in total link accidents over time depends on two components: that which is attributable to growth of traffic, and that which would occur if there were no change of flow. We shall refer to this latter trend as the underlying trend in accident risk. As stated above, it is believed to be due to changes in vehicle design and driver behaviour. It also incorporates minor improvements in road layout which occur from time to time but which fall short of major design changes.

It is important to allow for these trends in the accident-predictive models because if this is not done the models will over-estimate, by a greater and greater amount as time goes by, the number of accidents to be expected on a given stretch of road.

13.2 Incorporating accident trends in the models

The basic model (described in greater detail in Section 14) relating accident frequency on a link to traffic, link length and design features is:

$$A = k.Q^{\alpha}.L_L^{\beta}.\exp(\text{design features}) \qquad (13.1)$$

where A is the accident frequency (accidents per year)

Q is the traffic flow (AADT)

L_L is the link length (km)

'design features' are variables and factors relating to the road design

k, α and β are the parameters estimated by the regression.

For a given link, whose length and design features are constant, the accident frequency will change over time if either the constant *k* or the traffic flow *Q* changes with time. In order to incorporate accident trend, we therefore replace *k* and *Q* in the basic model (equation 13.1) by time-dependent terms, to give:

$$A = k_0.\exp(\theta t).[Q_0.\exp(\gamma t)]^{\alpha}.L_L^{\beta}.\exp(\text{design features}) \qquad (13.2)$$

where k_0 is the base-year accident frequency parameter

Q_0 is the base-year traffic flow

θ is the underlying trend in accident risk

γ is the traffic growth parameter

t is the difference (in years) between the modelled year and the base year

and the other terms are as before. In section 7.2, the average value of the traffic growth parameter for the single-carriageway schemes was estimated as 4.5 per cent per annum.

Equation (13.2) can be expressed as:

$$A = A_0.\exp[(\alpha\gamma + \theta)t] \qquad (13.3)$$

where A_0 is the accident frequency in the base year. This clearly shows the two components of the total trend in accident frequency: traffic growth ($\alpha\gamma$) and underlying trend (θ). In general, α is less than unity, so the dependence of accident numbers on traffic is non-linear, and the trend in accidents due to the traffic component is less than the growth rate in traffic.

The accident rate *R* is obtained by dividing the number of accidents in equation (13.2) by the vehicle-km $(Q.L_L)$ to give:

$$R = k_0.\exp(\theta t).[Q_0.\exp(\gamma t)]^{\{\alpha-1\}},$$

assuming $\beta = 1$, which is found to be (at least approximately) true in most cases. This can be re-arranged to give:

$$R = R_0.\exp[(\{\alpha-1\}\gamma + \theta)t] \qquad (13.4)$$

where R_0 is the accident rate in the base year. This clearly shows that the trend in accident rate over time also depends on two components: the same underlying trend (θ) as for accident frequency, and the *non-linear part* of the dependence of accident frequency on traffic growth ($\{\alpha-1\}\gamma$). If the dependence of accident frequency on traffic flow is linear ($\alpha = 1$), the latter term vanishes. If not, the accident rate is not independent of traffic level, but decreases as traffic increases.

In addition to these variations due to traffic growth and underlying trend, any change in the average value of the design factors over time will result in a change in accident frequency. If the design features are not explicitly included in the model, this may appear as a dependence of accident frequency on the opening year of the scheme. For this reason, a reduction in the average accident rate over time could result, at least partially, from the gradual replacement of older roads by roads of more modern design.

13.3 Estimation of the trend parameter

The underlying trend parameter θ was estimated by modelling. The procedure was to model the number of accidents occurring on each link in each individual year, taking account of variations in traffic flow, using a simple model based on the equation:

$$A_{it} = c_i.Q_{it}^{\alpha}.\exp(\theta t) \qquad (13.5)$$

where A_{it} is the number of accidents on link *i* in year *t*, Q_{it} is the traffic on link *i* in year *t*, and c_i is a link-specific constant into which are subsumed all the geometric and other variables pertaining to link *i* which remain constant over time. The other symbols have the same meaning as before.

This simple model essentially fits an exponential trend to the year-by-year numbers of accidents on each link and averages the trends over all links. It distinguishes between cases where the number of accidents on a link in a given year was zero, and cases where no data were available for a given year; the former contribute to the estimation but the latter do not.

Having deduced a value of the time-trend parameter by this method, the other parameters in the time-dependent model (equation 13.2) were estimated using the total number of accidents which occurred over the whole time period for which accident data were available. The procedure is described in Section 14. The remainder of this Section describes the results from the estimation of the time-trend parameter and their implications.

13.4 Results of time trend analysis for single carriageways

The value of θ obtained from this analysis was 0.0173. This implied an increase in the accident risk factor k of 1.7 per cent per year, but the standard error of 0.0144 means that this value was not significantly different from zero at the 5 per cent level.

The value of α was 0.82, and this was significantly different from 1 at the 5 per cent level. It was similar to that obtained from models with no time trend parameter.

13.5 Variation of time trend with scheme age

In section 11.3.1 it was found that link accidents on the pre-1979 schemes increased at 3.2 per cent per annum compared to a traffic growth rate of 4.5 per cent, implying a 1.3 per cent *reduction* in accident risk factor. Here, however, we have deduced a 1.7 per cent *increase*.

This anomaly was investigated by developing models using only the pre-1979 schemes, which gave a value of θ similar to the 1.3 per cent decrease. There are several possible explanations for this difference between the all-schemes model and that for the pre-1979 schemes:

- old schemes might indeed have a different trend (as well as possibly a different risk factor) from newer schemes;
- the trend might be common to all schemes but represent a decrease in early years (which would affect only the older schemes) and an increase in later years;
- traffic growth could be higher in later years than earlier, leading to a deceptively high number of accidents in later years, compared to the number expected if traffic growth were constant as represented in the model. The difference would appear in the trend term.

The effect of scheme age on numbers of accidents was investigated by including a term X^{δ} in the model, where X was the opening year of the scheme (referred to a base year of 1990) and δ was a model parameter. The value of δ was found to be -0.0189, with a standard error of 0.0058. This was a statistically significant result, and implied a 1.9 per cent per year decrease in accident risk with opening year. However, when opening year was included as an explanatory variable in the models along with design features of the road, it was found that the opening year became non-significant. This implied that it was the additional safety features incorporated in the design of newer roads which led to a reduced accident risk.

It was not possible to investigate the changes in time trend with scheme age on single carriageways further within the resource constraints of this study. However, in the dual-carriageway study some comparisons between single and dual carriageways were carried out, which indicated that there were no significant differences between singles and duals in each age group. Any apparent differences in trends between singles and duals could be ascribed to the larger proportion of newer schemes in the singles study. This work is described more fully in the report of the dual-carriageway study (Walmsley, Summersgill and Payne, 1998).

13.6 Conclusion from time trend analysis

The conclusion from this investigation of time-dependent accident risk was that there was no significant time trend effect on single-carriageway roads. Any apparent variation of accident risk with time was due to newer schemes having a lower risk than earlier schemes. It was therefore not necessary to include time trends in the models. It was, however, necessary to take account of the fact that traffic levels change over time.

In line with these conclusions, the models derived in this study and listed in Section 15 assumed an underlying accident trend of zero. In circumstances where the models are to be applied to a new single-carriageway road scheme, a similar trend of zero should be used. However, where single- and dual-carriageway models are required to be on a comparable basis, it should be noted that the sampling errors and distributions of scheme ages were such that the time trends found in the singles and duals studies were not inconsistent. It is therefore better to assume a common trend of -2.0 per cent per annum for both types of road. There was no statistically significant difference between a trend of zero and one of -2.0 per cent.

14 Regression analysis method

14.1 General description

The objective of the analysis was to relate the accident frequency (the average number of accidents per year) on the minor links to a range of explanatory variables, thus providing a model for examining the effect of vehicle flow and link characteristics. Such a model might also be used for predicting site-specific mean accident frequencies, for example when a new road scheme was being planned.

The statistical method used was a form of multiple regression analysis and was the same as that employed in a number of previous accident studies: accidents at four-arm roundabouts (Maycock and Hall, 1984), at rural T-junctions (Pickering, Hall and Grimmer, 1986), at four-arm single-carriageway urban traffic signals (Hall, 1986), at urban mini-roundabouts (Kennedy *et al*, 1998), at three-arm priority junctions on urban single-carriageway roads (Summersgill *et al*, 1996), three-arm single-carriageway urban traffic signals (Taylor *et al*, 1996), four-arm priority junctions (Layfield *et al*, 1996), and non-junction accidents on urban single-carriageway roads (Summersgill and Layfield, 1996). The same method was used in other TRL studies of accidents on modern rural trunk roads (Walmsley, Summersgill and Payne, 1998; Walmsley and Summersgill, 1998).

The explanatory or independent variables of the regression are the traffic flow and the geometric and other characteristics of the schemes and minor links. Since, however, the numbers of accidents in a given period do not follow a Normal distribution and, in particular, do not have a constant variance, classical least squares regression should not be used. Instead, the generalised linear modelling technique available in the computer program GENSTAT (Alvey *et al*, 1977) was used. This program allows the dependent variable in the regression analysis to be drawn from any of a family of distributions, in particular the Poisson distribution, and also allows for transformations of the variables in order to reduce a more complicated model to a linear form.

14.2 The form of the models

14.2.1 Link length and vehicle flow

The modelling process used relationships that have been found to be successful in similar studies. The basic form of the model relating the frequency of link accidents to traffic flow is:

$$A = k.Q^{\alpha}.L_L^{\beta} \qquad (14.1)$$

where A is the accident frequency (accidents per year) on the link

Q is the traffic flow (AADT) in vehicles per day

L_L is the length of the link (km)

k, α and β are parameters of the model.

If the value of β were found to be close to, and not different statistically from, 1, the model could be simplified to:

$$A = k.Q^{\alpha}.L_L \qquad (14.2)$$

This simple and comprehensible result would imply that the accident frequency *A* was a function of vehicle flow and was directly proportional to the length of the link. However, in the initial models tested in this study, the value of β was generally in the region of 0.85, and was statistically different from 1 at the 5 per cent level. β was therefore left as a coefficient in the models. The interpretation of this result is that there are more accidents per km on short links than on long ones, which could be due to the influence of junctions on the adjacent links.

14.2.2 Road layout features

Road layout features are introduced into the models using exponential terms. A multiplier of the form exp $[g_1G_1 + g_2G_2 + ...]$, where g_i is the coefficient of the variable G_i, representing a design feature of the road, is introduced into the model equations.

The form of the basic link model then becomes:

$$A = k.Q^{\alpha}.L_L^{\beta}.\exp[g_1G_1 + g_2G_2 + ...] \qquad (14.3)$$

In order to test the effect of the main features of the links on accidents, it is necessary to separate the features into those which can be measured on a continuous scale, such as bendiness, and those which group the data into two mutually exclusive subsets, such as the presence or absence of a hardstrip. The former produce a *variable* which is entered into the model as an exponential term, for example *exp[g_i.bendiness]*. The latter are entered into the model by defining a two-level *factor* which has a value of 0 for links without the feature and 1 for those with it. The addition of a factor to the model provides parallel regressions for each level of the factor, that is, separate values of the constant *k*, whilst sharing common values of the other parameters. The effect of including a 2-level factor is to add a multiplier *exp[g_i]* to the model for the higher level of the factor, for example for roads with hardstrips. This can be expressed algebraically by adding a dummy variable (taking only the values 0 or 1) to the model. In some cases, it is appropriate to use factors with 3 or more levels.

The form of the model then becomes:

$$A = k.Q^{\alpha}.L_L^{\beta}.\exp(a_1A_1 + a_2A_2 + ...).\exp(d_1D_1 + d_2D_2 + ...) \qquad (14.4)$$

where a_i is the coefficient of the variable A_i, and where D_i is a dummy variable relating to the higher level(s) of a factor and d_i is the coefficient estimated by the regression giving the difference from *k* of the constant for the second level of the factor.

Interactions between factors could be included in the same way to provide different constants for combinations of levels of the factors. Interactions between variables, or between a variable and a factor, could also be added to permit non-parallelism, that is, to provide separate flow exponents as well as separate constants for each level of the factors.

14.2.3 Using the models

Section 15 presents the results of the regression analysis. The tables of results give the values of the accident frequency parameter k_0 and the parameters for traffic flow, length and road layout variables. Strictly speaking, k_0 is the value appropriate to the base year and should be adjusted for the accident trend for other years, but for single-carriageway roads the trend was not significantly different from zero, so it is not necessary to make a correction.

14.3 Calibration of the models

In order to obtain the best estimate of the model parameters (in the jargon, to calibrate the model), it is desirable to use data from all the years for which accident data are available. For the basic model (equation 14.1), this total number of accidents is given by:

$$N_T = A.T = k.T.Q^{\alpha}.L_L^{\beta} \qquad (14.5)$$

where N_T is the total number of accidents occurring in T years, and the other variables are as before. Since N_T is a number which is randomly distributed around a mean value, it can be regarded as having a Poisson error distribution, which can be modelled using generalised linear modelling techniques.

14.3.1 Incorporation of time trends in the calibration

We know from Section 13 that both k and Q are time-dependent, so the accident frequency A is not constant. The total number of accidents is therefore not simply the product of A and T, but is given by integrating A with respect to t over the years T:

$$N_T = \quad Adt \qquad (14.6)$$

$$= \quad k_0.\exp\ (\theta t).[Q_0.\exp(\gamma t)]^{\alpha}.L_L^{\beta}.dt$$

where k_0, Q_0 and γ are as defined in section 13.2.

This form of relationship is not amenable to generalised linear modelling, but an acceptable approximation is:

$$N_T \quad A_m.T = k_0.\exp\ (\theta m).Q_m^{\ \alpha}.L_L^{\beta}.T \qquad (14.7)$$

where T is the number of years in the period for which accident data are available, A_m is the mean accident frequency over that period, m is the value of t at the midpoint of the period and Q_m is the weighted average traffic flow over the period, given by $Q_0.\exp(\gamma m)$. Where accident data are missing for one or more years in the period, m is taken as the centroid of the period rather than the midpoint.

When design variables are included, the model becomes:

$$\begin{aligned} N_T \quad A_m.T = \ & k_0.\exp\ (\theta m).Q_m^{\ \alpha}.L_L^{\beta}.T. \qquad (14.8) \\ & \exp(a_1A_1 + a_2A_2 + ...). \\ & \exp(d_1D_1 + d_2D_2 + ...) \end{aligned}$$

14.3.2 Transformation to linear form

Before fitting, the model (as given in equation 14.8) is transformed to the linear form using the standard log transformation to give:

$$\begin{aligned} \ln(N_T) = \ & \ln(k_0) + \theta m + \alpha.\ln(Q_m) \qquad (14.9) \\ & +\beta.\ln(L_L) + \ln(T) \\ & + (a_1A_1 + a_2A_2 + ...) \\ & + (d_1D_1 + d_2D_2 + ...) \end{aligned}$$

The model is then fitted by performing a regression of the dependent variable $ln(N_T)$ against the independent variables $ln(Q_m)$, $ln(L_L)$, A_i and D_i, including as an offset (constant) term. The results of fitting the model are the constant term $ln(k_0)$ and the coefficients α, β, a_i and d_i.

For single-carriageway roads the value of θ was found to be not significantly different from zero, so the term θm can be omitted from the offset. However, it is still necessary to include $ln(T)$ as an offset to take account of the growth in traffic.

14.4 Significance testing

The aim of the modelling is to obtain the best trade-off between the number of variables included in the model (keeping the number as small as possible to make interpretation easier) and the ability of the model to represent the data (keeping the fit as good as possible). The criterion used is to include in the model all those, and only those, variables which make a statistically significant improvement to the fit.

Each model is fitted in a step-by-step procedure, starting with a null model which simply fits the mean value of the dependent variable. In linear regression, the normal method of testing is to compare the mean deviance (variance divided by degrees of freedom) for the variables being tested against the mean deviance for the residual variance, that is, the variance which remains unexplained, using the F-ratio test. However, in circumstances where the error distribution is not Normal, as in this study, this method is not appropriate.

Instead, at each step the statistic calculated is the *scaled deviance* which gives a measure of the goodness of fit of the current model relative to the full model which fits all the data points exactly. Thus the smaller the scaled deviance becomes the better is the fit of the model to the data, reaching zero for a perfect fit.

A simple approach to the analysis assumes that the accident numbers on a particular link follow a Poisson distribution. In using the Poisson distribution, provided the predicted mean value of accidents in the study period is greater than about 0.5 (see Maycock and Hall, 1984), the scaled deviance is asymptotically distributed as χ^2 with $(n\text{-}p\text{-}1)$ degrees of freedom (where n is the number of data points and p the number of independent variables fitted), and may be used as a test of the goodness of fit of the model.

The significance of adding one or more terms to a model also needs to be assessed. Generally, the difference in scaled deviance between two nested models with degrees of freedom df_1 and df_2 is distributed like χ^2 with $(df_1 - df_2)$ degrees of freedom, so for the addition of one term, a value of at least 3.8 is required for significance at the 5 per cent level.

The Poisson assumption takes account only of the *within-site* variation of accident numbers, that is, the variation that occurs between successive samples of accidents taken from the same site. The accidents in this study, however, occur at a large number of links with different mean accident frequencies and densities. This adds an additional component of variation called *between-site* variation. The aim of the regression analysis is to explain as much as possible of this between-site variation. In general, however, some will remain. The effect is to make the variance-to-mean ratio for the accident numbers greater than unity (the ratio is unity for a Poisson distribution) and is known as *over-dispersion*. A further complication is that when accidents are broken down into groups, the mean number of accidents per link in the study period can be less than 0.5, and this reduces the scaled deviance below that expected for χ^2. The problems of over-dispersion and low mean values have been discussed by Maycock and Maher (1988).

In the analyses presented in this paper, a quasi-likelihood method is used to take account of over-dispersion in the presence of low mean values. The procedure is as follows. Each model is initially calculated assuming a Poisson distribution of accidents which has a variance-to-mean ratio (the *scale factor*) of unity. The amount of over-dispersion is then estimated by calculating the ratio of the generalised Pearson χ^2 function to the number of degrees of freedom *df* for that model. This provides a revised estimate of the scale factor *s* which can be used to recalculate the model. The model parameters themselves are unchanged, but both the scaled deviance and the standard errors of the parameters are affected by *s*. The addition of one term requires a scaled deviance drop of 3.8*s* and the true standard errors are estimated by multiplying the Poisson model standard errors by . In the results presented in this report the standard errors refer to a Poisson model and estimates of the scale factors are given.

14.5 Modelling procedure

This section gives some further details of the generalised linear modelling procedure.

14.5.1 Stepwise selection

All modelling was carried out using the GENSTAT package (Alvey *et al*, 1977), with the minor link as the unit of analysis.

The models were developed using a form of forward selection procedure on a pool of variables and factors to be tested. The significance test used was the drop, or rise, in scaled deviance produced by adding a term to, or removing a term from, the model. Variables and factors were added to, or dropped from, the models if the deviance drop, or rise, was greater than 3.8 times the scale factor (5 per cent significance for a change of 1 term).

The first step in the procedure was to test the effect of each variable individually, and to rank them in order of statistical significance. Variables and factors were then sequentially tried in the models, the most significant first, and were added to the model if this produced a significant drop in scaled deviance.

At each stage, whenever a new term was added to the model, the contributions of the existing terms in the model were checked, and terms were dropped if they had become non-significant. The process was repeated until no more terms could be added or dropped.

14.5.2 Significance testing for main parameters

A logical difficulty presents itself in deciding how to test the main parameters of the models. Normally, the statistical test applied is to test whether a particular parameter is significantly different from zero. This is the procedure used for features of the road such as bendiness; a statistically significant difference means that the feature has an effect on accidents. In the case of the main parameters for link length and traffic levels, however, we expect some dependence. The question to be answered is whether this dependence can be taken to be linear (that is, that accident rate or accident density is constant), or whether a more complex relationship must be used. In statistical terms, the null hypothesis is that α (the traffic flow parameter) and β (the link length parameter) are unity, and we test these parameters for significant differences from unity.

The results from the main models (Level 1, and Level 2 for accident classes) showed that α(the traffic flow parameter) and β (the link length parameter) were significantly different from 1. This was consistent with results from similar studies such as the dual-carriageway study (Walmsley, Summersgill and Payne, 1998). In some of the Level 2 accident group models, especially those with low numbers of accidents, one or both parameters were not significantly different from 1.

14.5.3 The effect of uncertainties in the explanatory variables

The generalised linear modelling technique that was used in the development of the models assumed that the values of all the explanatory variables were precisely known. But some of the variables that were tested in the models were estimates which contained uncertainty, in particular, where variables such as verge width were estimated from video measurements. The effect of ignoring such uncertainties would be to introduce bias into the estimation of the model parameters. The extent of the bias cannot be precisely known.

The earlier studies (Maycock and Hall, 1984 and others) identified a similar problem of uncertainty in the traffic flow estimates, and concluded that there was no existing procedure for properly analysing such data. TRL therefore let a small extra-mural contract with the University of Sheffield to develop a suitable procedure. This produced computer packages based on GLIM and GENSTAT which used iterative procedures and which took account of the uncertainties in the flow estimates to eliminate bias in the models.

Unfortunately, these packages could not be used to handle the more complex forms of model that were developed during this project. They did, however, indicate the expected extent of bias in the models parameters, which was found to be small (less than 10 per cent of the parameter value) and well within the quoted standard errors. This supports the assumption that any bias resulting from uncertainties in the explanatory variables, although present, is likely to be small.

14.6 Sequence of models

The general form of the models to be used for analysis was discussed above. This section describes the procedure for developing models for link accidents. It was similar to earlier studies carried out by or for TRL. The variables tested were similar to those tested in the dual-carriageway study (Walmsley, Summersgill and Payne, 1998), except where these referred to features of the dual-carriageway median. The unit of analysis was the minor link.

The regression modelling was undertaken in two main stages:

Level 1: relating total accident frequency on the minor links to various functions of the traffic flow and geometric variables,

Level 2: relating accident frequency on the minor links for each main accident group to various functions of the traffic flow and geometric variables. Level 2 models were also developed for each accident class (as defined in section 5.4).

At each stage, there were three sub-stages. In sub-stage A, the accident frequency was related to functions of the traffic flow and link length. In sub-stage B, the model was extended to include the principal geometric variables and factors. In sub-stage C, some further alignment and derived variables were examined.

The models which were developed were:

***Level 1: Total accident-flow models*:**

1A Total accidents as functions of traffic flow, number of years and link length

1B Total accidents as functions of traffic flow, number of years and link length, plus the following major design variables which are used in COBA-9:

Bendiness

Hilliness

Number of accesses

Mean verge width

Visibility

Carriageway width

Hardstrip width

Presence of kerb

Presence of continuous edge marking.

1C Total accidents as functions of traffic flow, number of years, link length and the above major design variables, plus the following extra design variables:

Speed according to the COBA-9 formula

Speed according to the COBA-10 formula

Design Speed

$V_{50,wet}$ (median speed of light vehicles on wet roads, used in calculating design speed)

Maximum curvature (minimum radius) on link

Proportion of link with radius of curvature in Range C

Proportion of link with radius of curvature in Range D or greater

Maximum gradient on link

Proportion of link with gradient greater than |6| per cent

Proportion of link with crest curvature greater than the absolute maximum advised in the standard

Proportion of link with sag curvature greater than the desirable maximum advised in the standard

Combinations of hills and bends: Proportion of link with radius less than 2880 metres and gradient greater than |6| per cent.

Level 2: Accident-flow models by accident group

Level 2 modelling is carried out for all groups. The unit of analysis is the minor link-side, ie each direction is analysed separately:

2A Accidents by group as functions of traffic flow, number of years and link length

2B Accidents by group as functions of traffic flow, number of years, link length and major design variables as in Level 1B, with the addition of the following directional variables:

Up-Hilliness

Down-Hilliness

Net gradient

Number of accesses on nearside of carriageway

Number of accesses on offside of carriageway.

Level 2C models were not developed because, of the extra variables introduced at Level 1C, only the speed variables were found to be significant. These, as explained in section 15.3.1, led to unstable models.

14.7 Presentation of results

In the tables of results, the Null model is presented first, consisting only of a constant term representing the mean number of accidents, followed by the best-fit Full model containing traffic and link length, and, where appropriate, design parameters. The difference in scaled deviance between the Null model and the Full model is a measure of the goodness of fit of the Full model, as discussed earlier. The column labelled Deviance Difference gives the drop in scaled deviance attributable to the given parameter, and gives an indication of the statistical significance of that parameter.

In assessing the usefulness of the significant variables in the model it is helpful to have an indication of their sensitivity over the range of the data. To do this the parameters are expressed in a multiplicative form in which each continuous variable is related to its mean value over all link sections. Thus, for example, if A_{mean} is the mean accident frequency when all variables take their mean values and A_{max} and A_{min} are the accident frequencies when one variable takes its maximum and minimum values, the tables give the multiplicative effect of each variable or factor in the full models as A_{max}/A_{mean} and A_{min}/A_{mean}.

15 Regression analysis results

This section presents the detailed results from the regression analysis models. The results are presented in the order described in Section 14.6, with results for Level 1 models (1A, 1B, 1C) followed by Level 2 models.

The tables of results give the values of the accident frequency parameter k, the traffic flow parameter α, the length parameter β, the coefficients of the continuous variables a_i and the coefficients for each level of the factors d_i. The models are presented in both exponent and logarithmic form for clarity of presentation. The user of the models must supply the traffic flow Q (AADT in vehicles per day) and the values of the continuous variables A_i, and decide which level of the factors D_i applies (e.g. whether the link has a hardstrip or not).

The user should also note that the tables give the value of k as k_0, the value appropriate to the base year (1990). For other years, this should be multiplied by exp(θt), where t is the year relative to 1990 (e.g. t=7 for 1997), as explained in Section 13. However, for single-carriageway roads only, this correction can be ignored, because the value of θ was not significantly different from zero. When comparisons between single and dual carriageways are to be made, the values of θ to be used is -0.020, so in the logarithmic models below, the value of $ln(k)$ should be taken as ($ln(k_0)$ - 0.020t), where $ln(k_0)$ has the value given in the Tables.

15.1 Level 1A models: Total accidents as function of traffic flow and link length

The general form of the model tested at Level 1A was:

$$A = k.Q^{\alpha}.L_L^{\beta},$$

or, in its logarithmic form:

$$\ln(A) = \ln(k) + \alpha.\ln(Q) + \beta.\ln(L_L),$$

where A = Accident frequency (accident per year)
Q = Traffic flow (AADT in vehicles per day)
L_L = Length of link
α = Model parameter for traffic flow
β = Model parameter for link length.

Table 15.1 presents the Level 1A results. The best fit model at Level 1A was:

$$\ln(A) = -8.407 + 0.833.\ln(Q) + 0.865.\ln(L_L),$$

or, in its exponent form,

$$A = 2.23E\text{-}4.Q^{0.833}.L_L^{0.865,}$$

where E-4 denotes the multiplier 10^{-4}. The indices α and β were both significantly different from 1 at the 5 per cent level, implying a non-linear relationship between accidents and flow. The non-linearity of the length parameter is thought to be due to a spill-over effect from junctions, whereby a certain proportion of the accidents on the link result from the proximity of the junctions at the ends of the link, as vehicles change lane, accelerate or decelerate. The proportion is likely to be smaller, the longer the link.

15.2 Level 1B models: Total accidents as function of traffic flow, link length, and major design variables

Table 15.2 shows the results from adding to the model deduced above, using a stepwise fitting procedure, the following variables: Bendiness, Hilliness, Number of accesses, Mean verge width, Visibility, Carriageway width, Hardstrip width, Presence of kerb, Presence of continuous edge marking.

The best fit Level 1B model was:

$$\ln(A) = -8.037 + 0.827.\ln(Q) + 0.923.\ln(L_L) + d1.HS + d2.WScheme,$$

or, in its exponent form,

$$A = 3.23E\text{-}4.Q^{0.827}.L_L^{0.923}.\exp(d1.HS + d2.WScheme),$$

where d1 = 0 for links with no hardstrip
d1 = -0.438 for links with hardstrips
d2 = 0 for standard-width (S2) schemes
d2 = -0.322 for wide (WS2) schemes.

The flow parameter α was significantly different from 1 (at the 1 per cent level), as was the length parameter β (at the 0.1 per cent level).

The results show that hardstrip width gave a significant improvement in fit; it reduced the scaled deviance by 31.4. A road with a 1-metre hardstrip would produce, other things being equal, an accident frequency 65 per cent of that on a road with no hardstrip. This effect was slightly larger than that found from the accident rate analysis in Section 12, where roads with a 1-metre hardstrip had an accident rate from the tabular analysis of 83 per cent of those without.

One rather surprising feature of the Level 1B results was that carriageway width did not appear as a significant variable, whereas its effect in the tabular analysis in Section 12 was very significant. The reason is that the latter analysis was on *schemes*, whereas the regression analysis was based on *minor links*. The carriageway width was re-calculated as a scheme average rather than a link average to give the factor *WScheme*, which was tested giving the results shown in Table 15.2. The scheme width factor *WScheme* was found to be significant at the 5 per cent level, and corresponded to a reduction in accident risk of 27 per cent on wide schemes as compared with standard schemes.

Investigation showed that there were 38 *WS2 links within otherwise S2 schemes*, compared with 46 *WS2 links within WS2 schemes*. These 38 isolated wide links had an average accident rate of 0.142 per Mveh-km, which was not significantly different from the average of

Table 15.1 Level 1A models: Total accidents as a function of traffic flow and link length

Model	*Model terms*	*Parameter value*	*s.e.*	*Deviance difference*	*Residual deviance*	*Degrees of freedom*	*Scale factor*
$A = k_0.Q^{\alpha}.L_L^{\beta}$							
Null	$\ln k_0$	-0.736	0.028		1866.0	428	6.00
Full	$\ln k_0$	-8.407	0.630		942.0	426	2.343
	α	0.833	0.069	228.0			
	β	0.865	0.035	695.6			

Table 15.2 Level 1B models: Total accidents as a function of traffic flow, link length and major design variables

Model	Model terms	Parameter value	s.e.	Deviance difference	Multiplicative effect at: Min	Mean	Max	Residual deviance	Degrees of freedom	Scale factor
$A = k_0.Q^{\alpha}.L_L^{\beta}.\exp[d1.HS + d2.WScheme]$										
Null	$\ln k_0$	-0.736	0.028					1866.0	428	6.00
Full	$\ln k_0$	-8.037	0.635					899.2	424	2.322
	α	0.827	0.069	228.0						
	β	0.923	0.037	695.6						
	HS	-0.438	0.071	31.4	1.36	1	0.88			
	WScheme	-0.322	0.099	11.4	-	-	-			

0.127 on S2 links. *Wide schemes*, in contrast, had an average accident rate of 0.095, which was significantly lower (at the 1 per cent level) than the average of 0.130 for S2 schemes.

This difference indicated that wide schemes had a low accident rate, but isolated wide links on an otherwise standard-width scheme would be expected to have an accident rate similar to the scheme as a whole.

Traffic levels accounted for a small part of the difference. Average traffic on WS2 schemes was 8 per cent higher than on S2 schemes. Since accidents increased as traffic to the power of about 0.8, instead of linearly, this would imply a 2 per cent lower accident rate on WS2 roads.

These conclusions are tentative, because there is an element of arbitrariness about the point at which an isolated link becomes long enough to be regarded as a scheme (or separable part of a hybrid scheme) in its own right. Further analysis would be required to investigate this further. In addition, there were only a small number of WS2 links on modern trunk roads on which to base the analysis. In order to increase the precision of the results, it would be necessary to extend the study to non-trunk roads.

Another unexpected result was that bendiness did not appear as a significant term. This seemed to imply that a bendy road was no less safe than a straight one, but there were other possible explanations:

i Bendiness might be an important effect for certain types of accident (say, vehicles leaving the road on bends), but not for the totality of accidents. This was tested in Level 2B.

ii Bendiness, which is a measure of the average curvature on a link, might not be sensitive enough to show an effect. A measure of the prevalence of sharp curves might be more sensitive. This was tested in Level 1C, using maximum curvature (minimum radius) and proportion of link with sharp curvature (small radius).

iii Bendiness might be significant only at larger values than those which occur on modern trunk roads. It would not be possible to test this effect without a sample of a different type of road.

iv Whilst increased bendiness at constant speed might increase accidents, drivers might well reduce speed as a reaction to the increased bendiness, so the net result would be neutral.

15.3 Level 1C models. Total accidents as function of traffic flow, link length and extra design variables

Table 15.3 shows the result of testing in the basic Level 1 model the extra design variables listed in Section 14.6.

15.3.1 Speed variables

It was not possible to test the speed variables in a stepwise regression, because they were highly correlated with each other, so the models were unstable. Instead, the variables were tested individually in the model, with the intention of retaining the ones which produced the greatest significant deviance drop.

The results (Table 15.3) show that $V_{50,wet}$ and Design Speed both produced a very significant drop in scaled deviance, that for $V_{50,wet}$ being larger. There were, however, signs of instability in the models, because the inclusion of $V_{50,wet}$ brought in a number of other variables - bendiness, visibility, carriageway width, verge width, opening year and edge treatment - as significant, whereas most of them did not appear to be significant in any other models. Most of these variables were used in the calculation of $V_{50,wet}$. In addition, the values of α and β changed substantially when speed was included. These were signs that speed was correlated with other variables. In the case of the COBA-9 and COBA-10 speeds, the reduction in scaled deviance was much less, COBA-10 speed being not significant at the 5 per cent level, and COBA-9 only just so.

In view of the fact that speed variables had either a low level of significance or produced unstable models, the usefulness of including speed as an explanatory variable was in some doubt. It was therefore decided not to retain speed variables in the models.

15.3.2 Other variables

The other extra design variables listed in section 14.6 were tested by stepwise regression as additions to the Level 1B model. No variables additional to those included at Level 1B were found to be significant.

15.3.3 Conclusion from Level 1C model

The conclusion was that including extra variables in the Level 1C model did not produce a significant and robust improvement on that for Level 1B.

Table 15.3 Level 1C models: Total accidents as functions of traffic flow, link length, and extra design variables

Model	*Model terms*	*Parameter value*	*s.e.*	*Deviance difference*	*Multiplicative effect at:* Min	*Mean*	*Max*	*Residual deviance*	*Degrees of freedom*	*Scale factor*
Level 1C Model using V50wet speeds										
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[s1.V50,wet + d1.HS +a1.Bendi +a4.Visib +a5.Lane +a6.Verge +a7.Xyear +a8.Edgetreat]										
Null	ln k_0	-0.720	0.028					1812.0	413	6.022
Full	ln k_0	12.16	1.44					620.9	403	1.511
	α	0.427	0.0722	233.0						
	β	1.145	0.0444	668.1						
	V50wet	-0.223	0.0139	136.3	31.47	1	0.22			
	HS	-0.257	0.0762	29.5	1.20	1	0.93			
	Bendi	-0.0113	0.0017	52.4	1.46	1	0.06			
	Visib	0.00313	0.0005	25.3	0.42	1	1.39			
	Lane	0.406	0.0864	18.1	-	-	-			
	Verge	0.110	0.0217	12.2	0.79	1	5.11			
	Xyear	0.0215	0.0068	9.7	0.75	1	1.13			
	Edgetreat	0.153	0.0622	6.0	-	-	-			
Level 1C Model using Design speeds										
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[s2.Designspeed + d1.HS + a1.Bendi]										
Null	ln k_0	-0.736	0.0277					1866.0	428	6.00
Full	ln k_0	-3.92	0.859					786.4	421	2.065
	α	0.585	0.0718	228.0						
	β	1.018	0.0394	695.6						
	Designspeed2	-1.949	0.536	109.1	-	-	-			
	Designspeed3	-1.058	0.528		-	-	-			
	Designspeed4	-2.289	0.554		-	-	-			
	HS	-0.402	0.0723	28.9	1.32	1	0.89			
	Bendi	-0.00604	0.0015	17.6	1.23	1	0.22			
Level 1C Model using COBA-9 speeds										
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[s3.COBA9 + d1.HS]										
Null	ln k_0	-0.736	0.0277					1866.0	428	6.00
Full	ln k_0	-3.08	1.61					900.1	424	2.317
	α	0.658	0.081	228.0						
	β	0.922	0.0368	695.6						
	HS	-0.371	0.0717	31.4	1.30	1	0.89			
	COBA9	-0.040	0.0125	10.5	1.55	1	0.77			
Level 1C Model using COBA-10 speeds										
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[s4.COBA10 + d1.HS]										
Null	ln k_0	-0.736	0.0277					1866.0	428	6.00
Full	ln k_0	-7.800	0.636					910.6	425	2.295
	α	0.796	0.0691	228.0						
	β	0.912	0.0366	695.6						
	HS	-0.404	0.0709	31.4	1.33	1	0.89			

15.4 Level 2 models: Accidents by accident group as function of traffic flow, link length and major design variables

In Level 2 each group of accidents was studied separately, and the influence of traffic, link length and design parameters on a particular type of accident was estimated. On the one hand, this was likely to reveal some effect of certain design features on accidents of a particular type which was concealed when all accidents were considered together; on the other hand, splitting the accidents into groups reduced the number of accidents available for study and inevitably reduced the likelihood of finding statistically significant results.

Modelling at Level 2 was carried out in 2 stages: Level 2A, where only traffic and link length were tested (with accident years included as an offset throughout); and Level 2B, where some of the major design features were included as detailed in section 14.6.

The results from all the Level 2 models are shown together in Table 15.4, and are discussed together for each accident group. Table 15.5 shows the results of modelling with two aggregated classes of accidents, namely Class 1: all single-vehicle accidents and Class 2: all multi-vehicle accidents. Note that the total numbers of accidents shown in each group may differ slightly from the totals in Table 11.5 because the latter contained a few

Table 15.4 Level 2 models: Accidents by accident group as a function of traffic flow, link length and major design variables

Model	*Model terms*	*Parameter value*	*s.e.*	*Deviance difference*	*Multiplicative effect at:* *Min*	*Mean*	*Max*	*Deviance*	*Degrees of freedom*	*Scale factor*
Group J: Pedestrian accidents (52)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-4.645	0.137					285.1	857	1.005
Full	ln k_0	-11.73	3.06					258.7	855	0.9238
	α	0.777	0.336	7.5						
	β	0.687	0.164	18.8						
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[a1.Bendi]										
Full	ln k_0	-11.24	3.04					254.0	854	0.9337
	α	0.675	0.335	7.5						
	β	0.813	0.175	18.8						
	Bendi	0.0116	0.0047	4.7	0.68	1	17.57			
Group K: Access accidents (49)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-4.704	0.141					279.7	857	1.141
Full	ln k_0	-2.73	3.03					276.7	855	1.139
	α	-0.217	0.338	0.2						
	β	0.25	0.153	2.7						
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[a2.Netgrad]										
Full	ln k_0	-3.22	3.04					270.0	854	1.198
	α	-0.169	0.338	0.2						
	β	0.273	0.154	2.7						
	Netgrad	0.263	0.0955	6.8	0.12	1	8.02			
Group L: Single vehicle, left carriageway nearside, LH bend (12)										
Too few accidents for effective modelling										
Group M: Single vehicle, left carriageway nearside, RH bend (17)										
Too few accidents for effective modelling										
Group N: Single vehicle, left carriageway nearside, straight road (134)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-3.699	0.086					563.8	857	1.426
Full	ln k_0	-10.71	1.95					459.4	855	1.002
	α	0.756	0.214	21.1						
	β	0.944	0.112	83.3						
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[a3.Hup]										
Full	ln k_0	-9.78	1.99					451.5	854	0.9953
	α	0.675	0.218	21.1						
	β	0.973	0.113	83.3						
	Hup	-0.0433	0.0168	7.9	1.31	1	0.04			
Group O: Single vehicle, left carriageway offside, LH bend (16)										
Too few accidents for effective modelling										
Group P: Single vehicle, left carriageway offside, RH bend (3)										
Too few accidents for effective modelling										
Group Q: Single vehicle, left carriageway offside, straight road (79)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-4.227	0.112					381.5	857	1.124
Full	ln k_0	-9.62	2.5					348.3	855	0.9608
	α	0.592	0.275	7.2						
	β	0.655	0.135	25.9						
$A = k_0.Q^{\alpha}.L_L^{\beta}$.exp[a3.Hup]										
Full	ln k_0	-8.67	2.55					343.9	854	1.013
	α	0.508	0.279	7.2						
	β	0.679	0.135	25.9						
	Hup	-0.0397	0.0202	4.4	1.28	1	0.05			

Table 15.4 (Continued)

Model	*Model terms*	*Parameter value*	*s.e.*	*Deviance difference*	*Multiplicative effect at: Min*	*Mean*	*Max*	*Deviance*	*Degrees of freedom*	*Scale factor*
Group R: Single vehicle, did not leave carriageway (59)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-4.519	0.129					304.4	857	0.9447
Full	ln k_0	-13.45	2.9					261.1	855	0.9006
	α	0.972	0.318	13.2						
	β	0.836	0.159	30.2						
Group S: Accident involving parked vehicle (37)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-4.984	0.16					237.7	857	1.184
Full	ln k_0	-18.67	3.81					188.5	855	1.047
	α	1.47	0.413	17.8						
	β	1.124	0.217	31.4						
Group T: 2+ vehicles, overtaking, same direction (82)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-4.19	0.11					396.7	857	1.174
Full	ln k_0	-9.71	2.45					340.2	855	0.917
	α	0.595	0.27	8.9						
	β	0.908	0.14	47.6						
	Hup	0.0225	0.013	2.9	0.87	1	5.4			
Group U: 2+ vehicles, overtaking, opposite direction, head-on (137)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-3.677	0.085					559.8	857	1.362
Full	ln k_0	-11.39	1.93					454.8	855	1.249
	α	0.834	0.212	24.7						
	β	0.911	0.11	80.3						
Group V: 2+ vehicles, overtaking, opposite direction, other collision (128)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-3.745	0.088					552.7	857	1.409
Full	ln k_0	-9.35	1.98					408.4	855	1.064
	α	0.582	0.219	16.7						
	β	1.254	0.121	127.5						
Group W: 2+ vehicles, no overtaking, same direction (182)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-3.392	0.073					736.9	857	2.43
Full	ln k_0	-17.19	1.7					602.4	855	1.382
	α	1.51	0.184	80.3						
	β	0.612	0.088	54.1						
Group X: 2+ vehicles, no overtaking, opposite direction (279)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	ln k_0	-2.965	0.059					846.0	857	1.501
Full	ln k_0	-10.55	1.36					596.0	855	1.19
	α	0.814	0.149	51.2						
	β	1.023	0.079	198.7						
Group Z: Single vehicle, involving overtaking (18)										
Too few accidents for effective modelling										

Table 15.5 Level 2 models: Accidents by accident class as a function of traffic flow, link length and major design variables

Model	Model terms	Parameter value	s.e.	Deviance difference	Multiplicative effect at: Min	Mean	Max	Deviance	Degrees of freedom	Scale factor
Class 1: Accidents involving a single vehicle (338)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	$\ln k_0$	-2.773	0.054					947.8	857	1.656
Full	$\ln k_0$	-9.53	1.22					736.8	855	1.163
	α	0.735	0.134	47.7						
	β	0.815	0.068	163.3						
$A = k_0.Q^{\alpha}.L_L^{\beta}.\exp[a3.Hup]$										
Full	$\ln k_0$	-8.69	1.24					721.4	854	1.13
	α	0.661	0.136	47.7						
	β	0.838	0.0684	163.3						
	Hup	-0.0363	0.0099	15.4	1.25	1	0.07			
Class 2: Accidents involving 2 or more vehicles (808)										
$A = k_0.Q^{\alpha}.L_L^{\beta}$										
Null	$\ln k_0$	-1.902	0.035					1779.0	857	3.015
Full	$\ln k_0$	-10.4	0.801					1122.0	855	1.582
	α	0.920	0.088	171.0						
	β	0.925	0.045	487.0						

minor links for which accident data were available but alignment data were not.

It should be noted that in many of the accident groups, the α and β parameters at Level 2A were not statistically different from 1. Some of the accident groups included so few accidents that meaningful results could not be produced; this is indicated in the Tables. Modelling was carried out only on link accidents (groups J to Z); groups A to I refer to junction accidents and were not modelled.

15.4.1 Group J: Accidents involving pedestrians

There were 52 accidents in the group. The best fit model was:

$$\ln(A) = -11.24 + 0.675.\ln(Q) + 0.813.\ln(L_L) + 0.0116.\text{Bendiness},$$

or, in its exponent form,

$$A = 1.31E\text{-}5.Q^{0.675}.L_L^{0.813}.\exp(0.01157.\text{Bendiness}).$$

The traffic flow and link length parameters were not significantly different from 1. The parameter for bendiness implies that a 10 degrees per km increase in bendiness increased accidents by 13 per cent. The addition of bendiness to the model reduced the scaled deviance from 258.7 (Level 2A model) to 254.0, representing only a small improvement in fit.

15.4.2 Group K: Accidents at accesses

There were 49 accidents in the group. The best fit model was:

$$\ln(A) = -3.22 - 0.169.\ln(Q) + 0.273.\ln(L_L) + 0.263.\text{Net gradient},$$

or, in its exponent form,

$$A = 0.0400.Q^{-0.169}.L_L^{0.273}.\exp(0.2625.\text{Net gradient}).$$

The parameter for net gradient implied that a 1 per cent increase in net gradient increased accidents by 30 per cent. The addition of net gradient to the model reduces the scaled deviance from 276.7 (Level 2A model) to 270.0, representing only a small improvement in fit. In this model, α was less than zero, in other words, accident numbers would decrease as traffic increased. This result could arise because busier roads tend to have fewer accesses; to test whether this conjecture was correct it would be necessary to include the number of accesses per km in the model.

15.4.3 Group L: Accidents involving a single vehicle leaving the carriageway on the nearside on a left hand bend.

There were 12 accidents in the group, which was insufficient for effective modelling.

15.4.4 Group M: Accidents involving a single vehicle leaving the carriageway on the nearside on a right hand bend.

There were 17 accidents in the group, which was insufficient for effective modelling.

15.4.5 Group N: Accidents involving a single vehicle leaving the carriageway on the nearside on a straight road.

There were 134 accidents in the group. The best fit model was:

$$\ln(A) = -9.78 + 0.675.\ln(Q) + 0.973.\ln(L_L) - 0.0433.\text{Up-hilliness},$$

or, in its exponent form,

$$A = 5.66E\text{-}5.Q^{0.675}.L_L^{0.973}.\exp(-0.0433.\text{Up-hilliness}).$$

The traffic flow and link length parameters were not significantly different from 1. The parameter for up-hilliness implied that a 1 metre/km increase in up-hilliness decreased accidents by 4 per cent. The addition of up-hilliness to the model reduced the scaled deviance from 459.4 (Level 2A model) to 451.5.

15.4.6 Group O: Accidents involving a single vehicle leaving the carriageway on the offside on a left hand bend.

There were 16 accidents in the group, which was insufficient for effective modelling.

15.4.7 Group P: Accidents involving a single vehicle leaving the carriageway on the offside on a right hand bend.

There were 3 accidents in the group, which was insufficient for effective modelling.

15.4.8 Group Q: Accidents involving a single vehicle leaving the carriageway on the offside on a straight road.

There were 79 accidents in the group. The best fit model was:

$$\ln(A) = -8.67 + 0.508.\ln(Q) + 0.679.\ln(L_L) - 0.0397.\text{Up-hilliness},$$

or, in its exponent form,

$$A = 1.72E\text{-}4.Q^{0.508}.L_L^{0.679}.\exp(-0.0397.\text{Up-hilliness}).$$

The traffic flow parameter was not significantly different from 1. The parameter for up-hilliness implied that a 1 metre/km increase in up-hilliness decreased accidents by 3 per cent. The addition of up-hilliness to the model reduced the scaled deviance from 348.3 (Level 2A model) to 343.9.

15.4.9 Group R: Accidents involving a single vehicle not leaving the carriageway.

There were 59 accidents in the group. The best fit model was:

$$\ln(A) = -13.45 + 0.972.\ln(Q) + 0.836.\ln(L_L),$$

or, in its exponent form,

$$A = 1.44E\text{-}6.Q^{0.972}.L_L^{0.836}.$$

The traffic flow and link length parameters were not significantly different from 1. No design parameters were significant.

15.4.10 Group S: Accidents involving a parked vehicle.

There were 37 accidents in the group. The best fit model was:

$$\ln(A) = -18.67 + 1.47.\ln(Q) + 1.124.\ln(L_L),$$

or, in its exponent form,

$$A = 7.79E\text{-}9.Q^{1.47}.L_L^{1.124}.$$

The traffic flow and link length parameters were not significantly different from 1. No design parameters were significant.

15.4.11 Group T: Accidents involving two or more vehicles travelling in the same direction, one overtaking.

There were 82 accidents in the group. The best fit model was:

$$\ln(A) = -9.71 + 0.595.\ln(Q) + 0.908.\ln(L_L) + 0.0225.\text{Up-hilliness},$$

or, in its exponent form,

$$A = 6.07E\text{-}5.Q^{0.595}.L_L^{0.908}.\exp(-0.0225.\text{Up-hilliness}).$$

The traffic flow and link length parameters were not significantly different from 1. Up-hilliness was significant at the 10 per cent level. Its coefficient was 0.0225, which implied that a 1 metre/km increase in up-hilliness increased accidents by 2 per cent. The addition of up-hilliness to the model reduced the scaled deviance from 340.2 (Level 2A model) to 337.3, representing only a small improvement in fit.

15.4.12 Group U: Accidents involving two or more vehicles travelling in opposite directions, one overtaking, head-on collision.

There were 137 accidents in the group. The best fit model was:

$$\ln(A) = -11.39 + 0.834.\ln(Q) + 0.911.\ln(L_L),$$

or, in its exponent form,

$$A = 1.13E\text{-}5.Q^{0.834}.L_L^{0.911}.$$

The traffic flow and link length parameters were not significantly different from 1. No design parameters were significant.

15.4.13 Group V: Accidents involving two or more vehicles travelling in opposite directions, one overtaking, not head-on collision.

There were 128 accidents in the group. The best fit model was:

$$\ln(A) = -9.35 + 0.582.\ln(Q) + 1.254.\ln(L_L),$$

or, in its exponent form,

$$A = 8.70E\text{-}5.Q^{0.582}.L_L^{1.254}.$$

The traffic flow parameter was not significantly different from 1. The link length parameter was just significantly different from 1 at the 5 per cent level. No design parameters were significant.

15.4.14 *Group W: Accidents involving two or more vehicles travelling in the same direction, no overtaking.*

There were 182 accidents in the group. The best fit model was:

$$\ln(A) = -17.19 + 1.51.\ln(Q) + 0.612.\ln(L_L),$$

or, in its exponent form,

$$A = 3.42E\text{-}8.Q^{1.51}.L_L^{0.6118}.$$

No design parameters were significant.

15.4.15 *Group X: Accidents involving two or more vehicles travelling in opposite directions, no overtaking.*

There were 279 accidents in the group. The best fit model was:

$$\ln(A) = -10.55 + 0.814.\ln(Q) + 1.023.\ln(L_L),$$

or, in its exponent form,

$$A = 2.62E\text{-}5.Q^{0.814}.L_L^{1.0226}.$$

The traffic flow and link length parameters were not significantly different from 1. No design parameters were significant.

15.4.16 *Group Z: Accidents involving a single vehicle and involving overtaking.*

There were 18 accidents in the group, which was insufficient for effective modelling.

15.5 Level 2 models: Accidents by accident class as function of traffic flow, link length and major design variables

In view of the low numbers of accidents in many of the groups tested in the previous section, Level 2 models were also developed for the accident classes defined in Section 5.4, that is, Class 1 (single-vehicle accidents) and Class 2 (accidents involving 2 or more vehicles). The Level 2 model results for accident classes 1 and 2 are shown in Table 15.5.

15.5.1 *Class 1: Accidents involving a single vehicle*

There were 338 accidents in the group. The best fit model was:

$$\ln(A) = -8.69 + 0.661.\ln(Q) + 0.838.\ln(L_L) - 0.0363.\text{Up-hilliness}),$$

or, in its exponent form,

$$A = 1.68E\text{-}4.Q^{0.661}.L_L^{0.838}.\exp(-0.0363.\text{Up-hilliness}).$$

The parameter for up-hilliness implied that a 1 metre/km increase in up-hilliness decreased accidents by 4 per cent. The addition of up-hilliness to the model reduced the scaled deviance from 736.8 (Level 2A model) to 721.4.

15.5.2 *Class 2: Accidents involving two or more vehicles*

There were 808 accidents in the group. The best fit model was:

$$\ln(A) = -10.40 + 0.920.\ln(Q) + 0.925.\ln(L_L),$$

or, in its exponent form,

$$A = 3.04E\text{-}5.Q^{0.9196}.L_L^{0.9248}.$$

The traffic flow and link length parameters were not significantly different from 1. This implied that single-vehicle accidents did not increase as rapidly with traffic levels as did multi-vehicle accidents. No design parameters were significant.

16 Summary and discussion

The previous section described the results for each regression model separately. In this section we draw together the results across all the models and the rate tabulations, and draw conclusions about the effect of various road features on accident frequencies.

Table 16.1 presents in summary form the tabulation analysis and Level 1 regression model results, and Table 16.2 the Level 2 model results. Two tables are presented for ease of handling, but they are in the same form, and should be read together. In general, the regression model results superseded those from the tabulations, which were a cruder form of analysis, but in some cases significant results were found in the tabulations which it was not possible to test in the models, so these are shown in the tables as an indication of a possible effect which was not fully tested.

16.1 Traffic flow

The Level 1 models suggested a traffic flow index α for total accidents in the region of 0.8. In other words, the relation between total accident frequency and traffic was not linear, so accident rate fell as traffic levels increased. Most of the Level 2 models gave similar values. In most of the models tested, the index was not significantly different from 1, so the evidence of this decrease was not firm, but cumulatively it was quite strong.

From the aggregate Level 2 models, the traffic flow index α for single-vehicle accidents was 0.66, and that for multi-vehicle accidents was in the region of 1, and could be greater than 1 as suggested by the result for accident group W (the only group for which α was significantly different from 1). This finding was consistent with earlier work (see section 3.2.4) which suggested that multi-vehicle accident rates increased, and single-vehicle accidents decreased, with increasing traffic flow.

16.2 Link length

From the single-vehicle aggregate, and the Level 1A model for total accidents, the link length index β was around 0.8, suggesting that, for a given traffic flow and design features, accident frequency was lower on longer links. This result could be attributed to a spill-over effect from adjacent junctions. However, again the evidence

Table 16.1 Summary of results from tabulation analysis and Level 1 models

	Symbol	*Unit*	*Link*	*Major link*	*Scheme*	*Major junc*	*Minor junc*	*Time trend model*	*1A*	*1B*	*1C V50, wet*	*1C Design speed*	*1C COBA-9*	*1C COBA-10*
Number of accidents			889	1400	1640	240	511	1296	1296	1296	1296	1296	1296	1296
Basic model variables														
Traffic	α	index			Assumed = 1			0.815	0.8325	0.8274	0.4272	0.5846	0.6584	0.7956
Link length	β	index			Assumed = 1				0.8654	0.9227	1.1451	1.0178	0.9219	0.9119
Time trend model variables														
Scheme Age	δ	year	-2.0	-1.8	-1.2	-5.0	-0.8	-1.9						
Time trend - pre-1979 schemes	θ	year	-0.3		-0.2			-1.3						
- all schemes								+1.7						
Major design variables														
Carriageway width	CW	WS2:S2	-27	-30	-21	-32	-2				+50			
Hardstrip	HS	Yes:no	-16	-17	-16	+24	+1			-35	-23	-33	-31	-33
Kerb	K	Yes:no	+14	+14	+12	-5	-8							
Scheme width	Wscheme	WS2:S2								-27				
Bendiness	B	deg/km									-1	-1		
Hilliness	H	m/km									-1			
No of accesses	NX	No/km												
Verge width	VW	m									+11			
Visibility	VISI	100m									+36			
Cont edge marks	CONEDGE	Yes:no									+17			
Years open		year									+2			
Additional design variables														
Speed		km/h									-20		-4	
Design speed (a)		85A:100B										+602		
Design speed (b)		120B:100B										-29		
Max curvature		10 deg/km												
% Range C curve		% of link												
% Range D+		% of link												
Max gradient		%												
% >|6|percent gradient		% of link												
% with sharp crest		% of link												
% with sharp sag		% of link												
Bends on hills		% of link												

1 *Shaded areas indicate which variables were tested - if no value is given, the variable was insignificant at the 5 per cent level.*

2 *Values shown are the percentage change in accident frequency for unit change in a continuous variable (eg an extra 10 degrees per km in bendiness, an increase of one-tenth in the proportion of link with a kerb, an additional access on a link) or for first category compared with second (eg width WS2 compared with S2)*

Table 16.2 Summary of results from Level 2 models

					Accident group															
					J	*K*	*L*	*M*	*N*	*O*	*P*	*Q*	*R*	*S*	*T*	*U*	*V*	*W*	*X*	*Z*
	Symbol	*Unit*	*1-vehicle*	*2+ vehicle*	*Pedn*	*Access*	*Left n/s LH bend*	*Left n/s RH bend*	*Left n/s no bend*	*Left o/s LH bend*	*Left o/s RH bend*	*left o/s no bend*	*other 1-veh*	*parked*	*o/take same*	*o/take head on*	*o/take other*	*no o/t same*	*no o/t opp*	*o/take 1-veh*
Number of accidents			338	808	52	49	12	17	134	16	3	79	59	37	82	137	128	182	279	18
Basic model variables																				
Traffic	α	index	0.661	0.9196	0.675	-0.169			0.675			0.508	0.972	1.47	0.595	0.834	0.582	1.51	0.814	
Link length	β	index	0.8379	0.9248	0.813	0.273			0.973			0.679	0.836	1.124	0.908	0.911	1.254	0.6118	1.0226	
Major design variables																				
Carriageway width	CW	WS2:S2																		
Hardstrip	HS	Yes:no																		
Kerb	K	Yes:no																		
Bendiness	B	deg/km			1.2															
Hilliness	H	m/km																		
No of accesses	NX	No/km																		
Verge width	VW	m																		
Visibility	VISI	100m																		
Cont edge marks	CONEDGE	Yes:no																		
Years open		year																		
Speed		km/h																		
Design speed (a)		85A:100B																		
Design speed (b)		120B:100B																		
Directional major design variables																				
Up-hilliness	N+	m/km	-4						-4			-3			+2					
Down-hilliness	N-	m/km																		
Net gradient	NG	%				+30														
No of nearside accesses	NXL	No/km																		
No of offside accesses	NXR	No/km																		

1 *Shaded areas indicate which variables were tested - if no value is given, the variable was insignificant at the 5 per cent level.*
2 *Values shown are the percentage change in accident frequency for unit change in a continuous variable (eg an extra 10 degrees per km in bendiness, an increase of one-tenth in the proportion of link with a kerb, an additional access on a link) or for first category compared with second (eg width WS2 compared with S2)*

was not clear; the indices in most of the other models were not significantly different from 1, and those that were did not show a consistent pattern.

16.3 Time effects

16.3.1 Variation of accident risk with time

The results from time trend analysis suggested that, on a link where traffic levels and design features did not change, accident frequency increased over time by 1.7 per cent per annum. However, this value was sensitive to the estimated trend in traffic levels and was not statistically significant. There was therefore no firm evidence of a trend in accident frequency with time, apart from that arising from changes in traffic levels.

16.3.2 Variation of accident risk with scheme age

The results from time trend analysis suggested that accident risk was correlated with scheme age, being lower for newer schemes. In the regression modelling, however, scheme age was not a significant variable. This implied that any correlation was due to features of the design of newer roads, rather than the newness of the roads as such. Overall, the accident rate on the sample of modern roads studied was 0.125 per MVkm. This was considerably lower than the COBA-9 rate of 0.23 per MVkm, which applied to roads of traditional design, and implied that modern roads were much safer. The accident rates for modern single carriageways used in the latest version COBA-10 were taken from this study.

16.4 Carriageway width, kerbs and hardstrips

16.4.1 Variation of accident risk with carriageway width

In the tabulation analysis, schemes with wide (WS2) carriageways had a significantly lower accident rate (0.095 per MVkm) than those with standard-width (7.3m) carriageways (0.131 per MVkm). In the regression analysis, the effect of carriageway width was significant when measured as a scheme average, with a WS2 scheme having a 27 per cent lower accident frequency than an S2 scheme, but not when measured as a link average. This difference indicated that wide schemes had a low accident frequency, but isolated wide links on an otherwise standard-width scheme would be expected to have an accident frequency similar to that of the scheme as a whole. The regression analysis also suggested that the lower accident rate on wide schemes was partly due to a correlation with traffic flow and presence of hardstrip.

Some wide carriageways appeared to have been formed by building a 7.3m road with hardstrips and then marking it as a 9.3m road without hardstrips. There were indications from the tabulation analysis that such roads had a lower accident rate than standard 7.3m roads with hardstrips, but the difference was not statistically significant. If this result were proved it would be important, as it would imply that it would be better to provide wider carriageways than a narrower carriageway with a hardstrip. However, this could not be done without further regression analysis, including interaction terms, which was not possible with the resources available for the study.

16.4.2 Variation of accident risk with presence of hardstrip

A consistent result from all the analyses performed was that the presence of a hardstrip reduced accidents significantly, compared to a road of the same width without hardstrip. The magnitude of the reduction varied between 16 per cent in the tabulation analysis and about 30 per cent in the Level 1 models.

16.4.3 Variation of accident risk with presence of kerb

There was an indication from the tabulation analysis that the presence of a kerb increased accident rate. However, presence of a kerb was not a significant variable in the regression analysis so the effect was not proven.

The higher accident rate for kerbed roads appeared to be associated with the absence of a hardstrip on standard width roads. Again, this effect could be investigated using interaction terms.

16.5 Curvature and gradient

Although it seems intuitively obvious that curvature and gradient should affect accident risk, there was little evidence of this from the analysis in this study. Curvature was statistically significant only in one small accident group, group J (pedestrian accidents) where bendiness was positively correlated with accidents. Gradient variables appeared in several models, in the form of up-hilliness which showed a decrease in accident risk of about 4 per cent per metre/km in accident groups N and Q (single vehicles leaving carriageway), and was also significant when all single-vehicle groups were aggregated. Gradient variables also featured in two other small accident groups, but with inconsistent values.

16.6 Speeds

The inclusion of COBA-9 and COBA-10 speeds in the models did not improve the fit greatly; COBA-10 speed was not statistically significant, and COBA-9 speed only just so. The inclusion of the mean speed $V_{50,wet}$ and its categorisation Design Speed was very significant. However, the model with $V_{50,wet}$ also included several other variables with unusual coefficients.

It is not clear why $V_{50,wet}$ improved the fit but the other speed variables did not. One possible reason is that $V_{50,wet}$ was correlated with other variables, being derived from them, but the same was true for the COBA-9 and COBA-10 speeds. Another possible reason is that the formulae were applied in some cases to very short links, instead of to links of at least 2 km long as specified in the standards. This would lead to spuriously high or low values. It was not possible to investigate these results in any more detail.

16.7 Other variables

A number of other variables, including many different measures of curvature and gradient, were tested in the models, as detailed in section 14.6. Apart from the variables described above, no other variables were found to give a significantly better fit to the data.

17 Conclusions

This section summarises the results from a substantive study of accidents on non-built-up single-carriageway trunk roads in England. The purpose of the study was to quantify the relationships between the numbers of injury accidents that occur, and the traffic and road layout variables that determine them, for single-carriageway trunk roads of modern design.

17.1 The study

The study covered 103 schemes, comprising most modern rural single-carriageway trunk road schemes in England which had opened since 1968. There were 2502 personal injury accidents on the schemes of which 1295 were link accidents, 391 occurred at major junctions and 816 occurred at minor junctions.

The main subject of this study was the development of predictive models of the annual numbers of accidents on a length of road in terms of the explanatory variables that affect them, using the technique of generalised linear modelling. These models allow the association between each statistically significant variable and accident risk to be determined, even where other variables are present and have their own effects, and they take account of any non-linear relationships between these variables and the accidents. The models ranged from simple whole-link models for total accidents, including only vehicle flow and link length as determining variables, to models for individual groups of accidents on which a wide range of layout variables were tested.

In order to provide a preliminary overview of the main variables which affect accidents, an extensive set of accident tabulations was prepared, a summary of which is presented in this report. These tabulations gave some useful insights into the characteristics of the accidents, and showed that accident rates were different on roads with different basic features, such as carriageway width and the presence of a hardstrip. These results gave a lead as to which variables should be included in the modelling.

17.2 Summary of results

The main conclusions from this study of accidents on modern dual carriageways are summarised below. The summary includes comparisons with the TRL study of modern rural dual-carriageway roads (Walmsley, Summersgill and Payne, 1998).

1 The average accident rate for link accidents on the schemes studied was 12.5 accidents per 100 MVkm. Rates for old rural A-roads of traditional design (pre-1968) were nearly twice as high (22 per 100 MVkm). The conclusion is that modern single-carriageway trunk roads of the kind studied in this report were safer on average than traditional A-roads. Therefore, replacing older roads with new ones would lead to a reduced accident risk. This result is consistent with the findings from the dual-carriageway study.

2 A consistent and statistically-robust result from all of the analyses performed was that the presence of a hardstrip reduced link accidents by about 20 to 25 per cent, other things being equal.

There was an indication that the presence of a kerb increased accident risk, but this appeared to be associated with the absence of a hardstrip so the effect of kerbs could not be identified separately.

In the dual-carriageway study, it was the absence of a kerb rather than the presence of a hardstrip which was found to reduce accident risk, but since most modern roads had kerbs or hardstrips but not both, the findings from the single- and dual-carriageway studies are consistent.

3 Wide (WS2, 10m) schemes had 27 per cent fewer accidents than standard-width (S2, 7.3m) schemes, other things being equal. In part, the lower accident rate on WS2 schemes was because such schemes tended to have higher traffic levels.

Isolated wide links on otherwise standard-width schemes would be expected to have an accident rate similar to the scheme as a whole. There was some evidence that the accident rate of such WS2 links was lower than that of S2 links when both had hardstrips and carried the same traffic, but this can only be regarded as a tentative result.

In the dual-carriageway study, there were too few 3-lane schemes to be able to draw any definite conclusions on the effect of carriageway width.

4 Horizontal alignment had no statistically significant effect on accidents, except for the small group of pedestrian accidents, in which bendiness was positively correlated with accidents. Gradient in the form of up-hilliness reduced single-vehicle accidents by about 4 per cent per metre/km, and this result was statistically significant.

In the dual-carriageway study, horizontal and vertical alignment had a small effect on total accidents, and a greater effect on certain accident groups. It must be remembered, however, that in both studies the range of curvatures and gradients was relatively small; there are no severe bends or hills on modern trunk roads.

5 The way in which the number of accidents on a link may vary over time depends on three components: that which occurs when there is no change of flow or of layout with time, that which is attributable to growth of traffic, and that which is attributable to changes in layout. The first (referred to as the underlying trend) is believed to be due to changes in vehicle design and driver behaviour. It also incorporates minor improvements in road layout which occur from time to time but which fall short of major design changes.

The underlying trend in link accident *frequency* over time when there was no change of traffic flow or of layout showed a small increase of 1.7 per cent per year. The trend in accident *rates* over time when there was no change of flow or of layout with time was the same 1.7 per cent per year. This result was not distinguishable statistically from no trend.

In the dual-carriageway study, the change with time was found to be a decrease of 2 per cent per year.

However, when sampling errors, and the larger proportion of older schemes in the dual-carriageway study, are taken into account, the results are not dissimilar.

6 After allowing for the above underlying trend and for growth in traffic, there remained a reduction in accidents of about 1 per cent per year which could be attributed to improvements in road design. As a result, the newest modern trunk roads, opened since the mid-1980s, were found to be safer than older (but still modern) roads opened between 1968 and 1980. The difference was due to the more widespread use of wide carriageways, hardstrips and other road design features which contribute to fewer accidents. This result is consistent with the findings from the dual-carriageway study.

7 The relation between link accident frequency and traffic flow was not linear. Traffic growth across all schemes averaged 4.5 per cent per year between 1979 and 1990, but because of this non-linearity, accidents increased by only 3.7 per cent per year. The non-linearity of accident *numbers* with traffic flow resulted in a decrease in accident *rates* of 0.8 per cent per year due to traffic growth. There was evidence that the non-linearity was greater for accidents involving a single vehicle and less for accidents involving more than one vehicle. These results are consistent with the findings from the dual-carriageway study.

8 The relation between total accident frequency and link length was also non-linear. A 10 per cent increase in link length would, for a given traffic flow and design features, give rise to an increase of about 8 per cent in accidents. This result could be attributed to a spill-over effect from adjacent junctions. This result is consistent with the findings from the dual-carriageway study.

   There was evidence that the non-linearity was less (and indistinguishable from linearity) for accidents involving more than one vehicle. This result is contrary to that found in the dual-carriageway study, where the non-linearity was less for single-vehicle accidents.

9 None of the other variables tested had a clear effect on accident risk. The reason why so few variables appeared to be significant is probably that this study was limited to single-carriageway roads of modern design. The range of values for most variables in the study, particularly curvature and gradient variables, was therefore limited by the highway design standards, so the sample of roads did not contain a wide enough range of values from which to deduce any effect.

Another reason for the lack of significant variables was the relatively low number of accidents which were available for analysis. In the dual-carriageway study, there were about four times as many accidents, because there were slightly more schemes with a greater total length and carrying more traffic, and because the schemes were on average older than in the single-carriageway study, giving more years of accident data for analysis. As a result, more variables were found to have a significant effect. However, as the single-carriageway study covered virtually every modern trunk road scheme and used all readily-available accident data, the coverage could not be increased.

17.3 In conclusion

The foregoing paragraphs summarise the features of single-carriageway roads which affect accident risk. Apart from these features, the results from the study show that, within the range found on modern roads, there is no significant variation in accident risk due to most of the design features tested, provided they are built to modern standards with hardstrips and with wide carriageways as appropriate to the traffic flow. With these provisos, there appear to be no major areas where a tightening of the standard would significantly improve accident risk. The variation allowed for in the standards does not appear to affect accident risk significantly.

18 Acknowledgements

The work described in this report was carried out in the Traffic and Transport Resource Centre of TRL under contract to the Traffic, Safety and Environment Division of the Highways Agency. The authors wish to thank Mr M Garnham for his advice and support throughout the project.

The authors are also grateful to DETR’s Highways Computing Division for supplying the tape masters and reference information for the alignment data, DETR Statistics Transport C Division, Regional Offices and County Councils for supplying traffic flow data, and the Safety and Environment Resource Centre at TRL for providing STATS19 records.

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Appendix 1: Single-carriageway schemes in the study

Scheme No.	*Road*	*Description*	*County*	*Date opened*	*Length km*
7.3m Standard width schemes (S2)					
1	A428	Bromham Bypass	Beds	Sep-86	3.7
2	A6	Elstow Bypass	Beds	Dec-82	1.5
3	A6	Silsoe Bypass	Beds	Feb-81	1.5
4	A10	Melbourn Bypass	Cambs	Jul-88	5.1
5	A45	St Neots Bypass	Cambs	Dec-85	4.7
6	A47	West Walton Bypass	Cambs	Jun-84	6.1
7	A47	Wisbech Bypass	Cambs	Jun-84	4.6
8	A10	Ely/Littleport Bypass	Cambs	Jun-86	12.4
10	A17	West of Kings Lynn to County Boundary	Norfolk	Dec-82	10.4
11	A47	East Dereham Bypass	Norfolk	Mar-78	7.6
12	A47	Great Yarmouth Western Bypass	Norfolk	Mar-86	1.9
13	A47	Swaffham Bypass	Norfolk	Jun-81	2.0
14	A10	Southery to South Runcton	Norfolk	Jun-80	16.4
15	A11	Attleborough Bypass	Norfolk	May-85	7.0
17	A50	Dove Bridge Improvement	Derby/Staffs	May-77	.7
18	A47	Billesdon Bypass	Leics	Oct-86	4.3
19	A47	Uppingham Bypass	Leics	Jun-82	3.0
20	A47	Wardley Hill Improvement	Leics	Oct-87	3.7
21	A52	Bottesford Bypass	Leics	Feb-89	4.7
22	A17	Heckington Bypass	Lincs	Dec-82	3.1
23	A17	Long Sutton to Sutton Bridge Bypass	Lincs	Oct-89	7.9
25	A17	Swineshead Bypass	Lincs	May-85	4.0
27	A46	Lincoln Relief Road (Southern section)	Lincs	Dec-85	6.1
28	A428	Little Houghton Bypass	Northants	Dec-79	3.1
29	A43	Broughton Diversion	Northants	Dec-84	3.1
30	A43	Bulwick Bypass	Northants	Apr-86	2.8
32	A57	Worksop Southern Bypass (Western section)	Notts	May-86	1.7
33	A57	Worksop Southern Bypass (Eastern section)	Notts	May-86	4.6
34	A66	Bowes Bypass	Durham	Jun-83	2.1
36	A1	Belford Bypass	Northumberland	Jul-83	5.6
37	A1	Berwick Upon Tweed Bypass	Northumberland	Oct-83	8.3
38	A1	Felton Bypass	Northumberland	Aug-81	4.3
39	A1	Warenford Diversion	Northumberland	Oct-77	1.1
40	A69	Greenhead Diversion	Northumberland	Dec-84	3.0
41	A1	Alnwick Bypass	Northumberland	Dec-85	9.6
43	A1	Haggerston Diversion	Northumberland	Apr-72	7.5
45	A500	Barthomley Link to M6	Cheshire	Jul-87	5.0
46	A51	Tarvin Southwest Bypass	Cheshire	Jun-84	2.7
47	A590	Arrad Foot Diversion	Cumbria	Nov-78	1.1
49	A590	Meathop to Sampool Bridge	Cumbria	Mar-82	2.8
50	A595	Thursby Bypass	Cumbria	Aug-87	2.5
51	A66	Peel Wyke to High Briery Improvment	Cumbria	Jun-77	9.7
52	A66	Troutbeck Diversion	Cumbria	Oct-82	1.5
53	A66	Troutbeck to Highgate Imprv	Cumbria	Nov-77	5.3
54	A66	Chapel Brow to Peel Wyke Improvement	Cumbria	Apr-77	16.8
55	A66	High Briery to Troutbeck Diversion	Cumbria	Aug-77	10.0
56	A59	Whalley and Clitheroe Bypass	Lancs	Apr-70	12.7
57	A21	Robertsbridge Bypass	E Sussex	Nov-89	2.1
58	A339	Kingsclere Bypass	Hants	Sep-82	2.8
59	A2	Lydden Bypass	Kent	Feb-77	1.2
60	A2	Lydden to Whitfield Diversion	Kent	Feb-77	3.0
61	A2	Whitfield to Dover Diversion	Kent	Feb-77	3.0
62	A423	Dorchester-on-Thames Bypass	Oxon	Nov-82	3.0
63	A38	Saltash Bypass (east) (Tunnel - not analysedfor kerb/hardstrip)	Cornwall	Sep-88	1.0
63	A38	Saltash Bypass (west)	Cornwall	Sep-88	1.2
64	A39	Marshgate Diversion	Cornwall	May-85	6.2
65	A39	St Columb Bypass	Cornwall	Jun-77	4.4
67	A39	Barnstaple Bypass	Devon	Jul-89	12.8
68	A39	Bideford Bypass	Devon	May-87	7.8
69	A361	North Devon Link Stage 2A	Devon	Nov-88	25.3
70	A361	North Devon Link Stage 2B	Devon	Jul-89	18.8
71	A31	Ferndown Bypass	Dorset	Dec-86	3.0
72	A31	Wimborne Bypass	Dorset	Sep-81	8.3
73	A31/35	Bere Regis Bypass	Dorset	Jul-82	2.0
74	A35	Bridport Link Road	Dorset	Jul-88	4.4

Scheme No.	Road	Description		County	Date opened	Length km
75	A35	Dorchester Bypass		Dorset	Oct-88	5.8
76	A40	Gloucester Northern Bypass		Gloucs	Jun-83	2.0
77	A40	Northleach Bypass		Gloucs	Jul-84	6.5
80	A417	Cirencester Eastern Bypass		Gloucs	Apr-74	.9
82	A36	Beckington Bypass (Northern section)		Somerset	Nov-89	.6
84	A36	Heytesbury Bypass		Wilts	Dec-86	1.7
85	A36	Steeple Langford Bypass		Wilts	Jan-89	2.0
87	A36	Black Dog Diversion		Wilts	Apr-74	3.1
90	A435	Sedgeberrow Bypass		Heref & Worcs	May-89	2.2
91	A49	Brimfield Bypass		Heref & Worcs	Mar-83	3.4
92	A49	Leominster Bypass		Heref & Worcs	Nov-88	6.3
93	A41	Newport Bypass		Shrop/Staffs	Oct-84	7.7
95	A49	Ludlow Bypass		Shrop	Jan-80	8.1
96	A49	Prees Bypass		Shrop	Aug-88	1.4
100	A1079	Beverley bypass (Southern section)		Humberside	May-80	4.6
101	A1079	Beverley Bypass (Northern section)		Humberside	May-80	2.9
102	A19	Riccall and Barlby Bypass		N Yorks	Oct-87	7.1
104	A629	Skipton Southwest Bypass		N Yorks	Oct-82	3.9
106	A65	Settle and Giggleswick Bypass		N Yorks	Dec-88	5.5
107	A57	Aston Relief Road (Eastern section)		S Yorks	Nov-84	1.0
108	A57	Aston Relief Road (Western section)		S Yorks	Nov-84	2.8
109	A616	Stocksbridge to M1		S Yorks	May-88	11.2
110	A660	Otley Bypass		W Yorks	Aug-84	2.1
10m Wide carriageway schemes (WS2)						
24	A17	New Washway Road		Lincs	Aug-88	5.2
31	A52	Bingham Bypass		Notts	Dec-86	2.9
35	A66	Darlington Bypass		Durham	Nov-85	8.5
66	A30	Polstrong to St Erth		Cornwall	Nov-84	9.9
81	A303	Ilminster Bypass		Somerset	Jul-88	13.5
89	A435	Evesham Bypass		Here & Worcs	Jul-87	6.1
94	A41	Hinstock Bypass		Shrop	Dec-83	1.8
97	A5	Oswestry Link		Shrop	Dec-86	6.4
98	A483/5	Oswestry Bypass		Shrop	Dec-86	10.6
Hybrid schemes (part S2, part WS2)						
9	A10	Buntingford Bypass	S2	Herts	Jun-87	2.2
			WS2			1.9
44	A49	Tarporley Bypass	S2	Cheshire	Sep-86	2.5
			WS2			1.2
48	A590	Haverthwaithe Diversion	S2	Cumbria	Nov-76	2.2
			WS2			1.6
86	A36	Warminster Bypass	S2	Wilts	Oct-88	7.6
			WS2			2.0
103	A59/65	Skipton Northern Bypass	S2	N Yorks	Oct-82	4.7
			WS2			1.1
105	A64	Seamer and Crossgates Bypass	S2	N Yorks	Feb-88	2.8
			WS2			1.2

Schemes rejected or otherwise amended for the reasons stated:

16	A6	Chapel-en-le-Frith Bypass	Derby	Aug-87	No video
26	A46	Lincoln Relief Road (Northern section)	Lincs	Dec-85	Not trunk road
42	A1	Cawledge Improvement	Northumberland	Apr-74	All dual carriageway
61	A2	Whitfield to Dover Diversion	Kent	Feb-77	Subject to major roadworks, scheme truncated
63	A38	Saltash Bypass	Cornwall	Sep-88	Central section dual carriageway. The remainder of the scheme was sub-divided into 63 (eastern section) and 63 (western section)
73	A31/35	Bere Regis Bypass	Dorst	Jul-82	Eastern section not trunk road, scheme truncated
78	A417	Birdlip Bypass	Gloucs	Dec-88	No video
79	A48	Chepstow Inner Relief Road	Gloucs	Jan-88	No HRM data
83	A36	Beckington Bypass (Southern section)	Somerset	Nov-89	Under construction at time of survey, scheme abandoned
88	A40	Ross-on-Wye Relief Road	Here & Worcs	Feb-85	No HRM data
99	A46	Stratford Northern Bypass	Warks	Jun-87	No video

Appendix 2: Coding scheme for accident types

	Code number	*Number of accidents*
Junction accidents		
Pedestrian accidents:		
1) 1 vehicle	001	14
2) 2 vehicles	002	3
3) 3+ vehicles	003	1
All other junction accidents:		
a) 1 vehicle		
1) Turning left	004	5
a. Leaving major	005	1
b. Entering major	006	6
c. Other		
2) Turning right	007	5
a. Leaving major	008	5
b. Entering major	009	11
c. Other	010	224
3) Other manoeuvres		
b) 2 vehicles		
1) One turning left		
a. Leaving major	011	15
b. Entering major	012	36
c. Other	013	26
2) One turning right	014	139
a. Leaving major	015	237
b. Entering major	016	94
c. Other		
3) One vehicle parked	017	5
4) Other manoeuvres, both same direction	018	107
5) Other manoeuvres, both opposite or different directions	019	182
c) 3+ vehicles	020	3
1) One turning left		
a. Leaving major	021	3
b. Entering major	022	7
c. Other		
2) One turning right		
a. Leaving major	023	27
b. Entering major	024	13
c. Other	025	11
3) One vehicle parked	026	1
4) Other manoeuvres, all same direction	027	15
5) Other manoeuvres, some opposite or different directions	028	11
Link accidents		
Pedestrian accidents:		
1) 1 vehicle	029	49
2) 2 vehicles	030	8
3) 3+ vehicles	031	3
Access accidents:		
a) Single vehicle		
1) Turning left		
a. Leaving major	032	0
b. Entering major	033	1
2) Turning right		
a. Leaving major	034	0
b. Entering major	035	0
b) Multi-vehicle		
1) One turning left		
a. Leaving major	036	1
b. Entering major	037	2
2) One turning right		
a. Leaving major	038	31
b. Entering major	039	16
All other link accidents:		
a) 1 vehicle		
1) Left carriageway nearside		
a. going ahead on left hand bend	040	12
b. going ahead on right hand bend	041	17
c. going ahead, other	042	134
2) Left carriageway offside		
a. going ahead on left hand bend	043	16
b. going ahead on right hand bend	044	3
c. going ahead, other	045	80
3) Did not leave carriageway		
a. going ahead on left hand bend	046	2
b. going ahead on right hand bend	047	2
c. going ahead, other	048	55
4) Other manoeuvres	049	0
b) 2 vehicles		
1) One vehicle parked	050	37
2) Overtaking involved		
a. Same direction		
i Rear end collision	051	18
ii Other collision	052	51
b. Opposite directions		
i Head-on collision	053	65
ii Other collision	054	65
3) Overtaking not involved		
a. Same direction		
i Rear end collision	055	105
ii Other collision	056	21
b. Opposite directions		
i Head-on collision	057	95
ii Other collision	058	112
c) 3+ vehicles		
1) One vehicle parked	059	0
2) Overtaking involved		
a. All same direction		
i Rear end collision	060	7
ii Other collision	061	7
b. Different directions		
i Head-on collision	062	71
ii Other collision	063	63
3) Overtaking not involved		
a. All same direction		
i Rear end collision	064	54
ii Other collision	065	2
b. Different directions		
i Head-on collision	066	44
ii Other collision	067	28
d) Single vehicle overtaking	068	18

Appendix 3: Amalgamation of accident types into groups

Group	*Code*	*Description*	*Number of Accidents*
Junction accidents			
A	1-3	Pedestrian accidents	18
B	4-10	1-veh accidents	257
C	11-13 and 20-22	2+ veh, left turn	90
D	14+23	2+ veh, right turn leaving major	166
E	15+24	2+ veh, right turn entering major	250
F	16+25	2+ veh, other right turn	105
G	17+26	2+veh involving parked vehicle	6
H	18+27	2+ veh, other manoeuvre, same direction	122
I	19+28	2+ veh, other manoeuvre, opp directions	193
Link accidents			
J	29-31	Pedestrian accidents	60
K	32-39	Access accidents	51
L	40	1-veh, left c/w nearside, LH bend	12
M	41	1-veh, left c/w nearside, RH bend	17
N	42	1-veh, left c/w nearside, straight road	134
O	43	1-veh, left c/w offside, LH bend	16
P	44	1-veh, left c/w offside, RH bend	3
Q	45	1-veh, left c/w offside, straight road	80
R	46-49	1-veh, did not leave c/w	59
S	50+59	Accident involving parked vehicle	37
T	51+52+60+ 61	2+ veh, overtaking, same direction	83
U	53+62	2+ veh, overtaking, opp dirn, head-on	136
V	54+63	2+veh, overtaking, opp dirn, other colln	128
W	55+56 +64 +65	2+ veh, no overtaking, same direction	182
X	57+58 +66 +67	2+ veh, no overtaking, opp direction	279
Z	-	1-veh, involving overtaking	18
Total junction accidents: 9 groups			
1 group		Pedestrian accidents	18
1 group		1-vehicle accidents	257
7 groups		2+ veh accidents	932
Total Junction accidents			1207
Total link accidents: 16 groups			
1 group		Pedestrian accidents	60
1 group		Access accidents	51
8 groups		1-vehicle accidents	339
6 groups		2+ veh accidents	845
Total link accidents			1295
Total accidents: 25 groups			
Total junction accidents			1207
Total link accidents			1295
Total accidents			2502

Appendix 4: List of variables measured

Alignment Data

Variable	*Minimum*	*Maximum*	*Mean*
Bendiness (°/km)	1.5	283.0	35.3
Maximum curvature (°/km)	4.0	996.6	118.0
equivalent to minimum radius of:	14400m	60m	485m
Proportion in range C[1]	0%	100%	26%
Proportion in range D or sharper[2]	0%	100%	22%
Hilliness (m/km)	0.3	17.3	12.4
Maximum gradient (%)	0.0%	14.4%	2.3%
Proportion > | 6 | %[3]	0%	100%	1.6%

1 Range C: 1020m to 2880m radius for design speed 100 km/h
2 Range D: 510m to 1020m radius; Departure: <510m radius, for design speed 100 km/h.
3 Desirable maximum gradient for single carriageway

Traffic Data

Variable	*Minimum*	*Maximum*	*Mean*
Average annual daily flow	2883	35904	10309

Link data - Continuous variables

Variable	*Minimum*	*Maximum*	*Mean*
Access left side	0	14	2
Access right side	0	14	2
Total no. accesses	0	17	3
Accesses per km	0	87.5	4.0
Visibility (m)	100	500	420
Lane width (m)	3.0	7.5	4.1
Hardstrip width (m)	0	1	0.7
Verge width (m)	0	30	3
Length of climbing-lane or local dualling (m)	0	7600	96
Lighting columns - number	0	45	1
Lighting columns - dist from carriageway (m)	0	4	0.1
Other columnar obstacles			
- type	0	6	1
- number	0	9	1
- distance (m)	0	9	1
Distance to obstruction (m)	0	35	3

Link data - Category variables

Variable	*Category*	*Number of minor links*
Land use at left of road	Not identifiable	476
	Woodland	369
	Farmland	19
	Developed	3
	Rough pasture/moorland etc	1
Speed limit	40 mph	10
	60 mph	858
Number of lanes in each direction	1	835
	2	33
Presence of warning signs	Absent	864
	Present	4
Road markings	None	2
	Broken white line	448
	Warning line	231
	Double lines:	
	- no overtaking in direction of travel	17
	- no overtaking in opposite direction	19
	- no overtaking in either direction	39
	Ghost island with broken edge lines	104
	Ghost island with solid edge lines	4
	Other	4
Edge treatment	None	19
	Kerbing only	164
	Hardstrips and kerbing	236
	Hardstrips and french drain	119
	Concrete channel and hardstrip	4
	Hardstrip only	304
	Other	11
	Lay-by abuts carriageway without an island	6
	Hardstrip, kerbing and french drain	5
Edge markings	None	77
	Continuous white line	749
	Broken white line	35
	Other	7
Obstruction severity	Severe	24
	Moderate	486
	Slight	317
	Very slight	41
Verge type	Footpath only	13
	Grass/grass and vegetation	626
	French drain and grass	162
	Small grass verge & footpath	32
	Lay-by, slip road or other hard surface only	9
	No verge present	2
	Other	19
	Lay-by & grass etc	5
Slope severity	Zero (0 ± 10°)	349
	Shallow (± 10° to ± 30°)	257
	Steep (± 30° or more)	252
	Exposed rock face	1
	Retaining wall	2
	Bridge parapet	5
	Road in tunnel	2
Slope length	None/negligible slope	254
	Short slope (4m or less)	262
	Medium slope (4m to 10m)	294
	Long slope (10m or more)	56
	Road in tunnel	2
Presence of safety fences	Absent	435
	Present	433
Presence of clearway signs	Absent	670
	Present	198

Junction data - Continuous variables

Variable	*Minimum*	*Maximum*	*Mean*
Main island size (m)	0	75	7
Length of stagger (m)	0	500	11
Circulatory width (m)	0	12	2
Entry lane width (m)	0	90	6
Exit lane width (m)	0	99	7
Length of hatching (m)	0		74
Length of hard island (m)	0		66
Length of two-lane section	0		49

Junction data - Category variables

Variable	*Category*	*Number of minor links*
Number of lanes at stopline	1	1088
	2	450
Signs present on central island at roundabout	None	316
	Keep left only	19
	Chevron arrow only	3
	Keep left & chevron arrow	363
	Other	2
	Present but not visible	84
	Not a roundabout	751
Markings 1	None	909
	Directional arrows only	296
	Route directions & arrows	7
	Speed suppression bar marks	2
	Slow	9
	Other (eg give-way)	273
	Not visible	42
Markings 2	None	1483
	Directional arrows only	1
	Speed suppression bar marks	1
	Other (eg give-way)	22
	Not visible	31
Pedestrian facilities at the junction	None	1425
	Refuge only with drop kerbs	85
	Pedestrian crossing with signals	5
	Other	14
	Drop kerbs only	9
Number of lanes in hatched section	0	798
	1	663
	2	76
	3	1
Number of lanes in hard island section	0	893
	1	292
	2	341
	3	12
Island type present on minor arm	None	542
	Small painted splitter	13
	Small hard splitter (<5m)	142
	Large hard splitter (>5m)	426
	Not minor arm	375
	Two small hard splitters	4
	Two large hard splitters	9
	One small & one large splitters	18
	One hard & one painted splitters	9

Appendix 5: Accident rates by scheme, 1986-90

													Number of accidents			Accident rates per MVkm			Juncn rates per Mveh	
Scheme	*Width*	*Strip*	*Kerb*	*Age band*	*Length km*	*Acc yrs*	*Total MVkm*	*Total Mveh*	*No of major jncs*	*No of minor jncs*	*MjJnc Mveh*	*MnJnc Mveh*	*Link*	*MjJnc*	*MnJnc*	*Link*	*MjLink*	*Sch*	*MjJnc*	*MnJnc*
1	S2	1	0	NEW	3.7	4	61.0	16.4	1.0	2.5	16.4	41.1	5.0	1.0	8.0	0.082	0.213	0.230	0.061	0.195
2	S2	1	0	MID	1.4	5	32.3	21.2	1.5	0.5	31.8	10.6	14.0	6.0	1.5	0.434	0.480	0.666	0.188	0.141
3	S2	1	1	MID	1.5	5	36.0	23.9	0.0	1.0	0.0	23.9	1.0	0.0	3.0	0.028	0.111	0.111	0.000	0.126
4	S2	1	0	NEW	5.1	2	46.4	9.1	0.0	6.0	0.0	54.3	9.0	0.0	3.0	0.194	0.258	0.258	0.000	0.055
5	S2	1	0	MID	4.7	5	152.9	32.3	0.0	1.5	0.0	48.4	13.0	0.0	18.5	0.085	0.206	0.206	0.000	0.382
6	S2	1	0	MID	6.1	5	132.7	21.9	1.0	3.5	21.9	76.8	26.0	7.0	3.0	0.196	0.218	0.271	0.319	0.039
7	S2	1	0	MID	4.6	5	87.9	19.3	0.5	2.5	9.6	48.2	9.0	8.5	0.5	0.102	0.108	0.205	0.881	0.010
8	S2	0	0	NEW	12.4	4	105.6	8.5	3.0	8.0	25.5	68.0	16.0	9.0	16.0	0.152	0.303	0.388	0.353	0.235
10	S2	1	0	MID	10.4	5	198.3	19.1	0.0	2.5	0.0	47.9	31.0	0.0	20.5	0.156	0.260	0.260	0.000	0.428
11	S2	1	0	OLD	7.6	5	116.8	15.4	0.0	5.0	0.0	77.0	27.0	0.0	4.0	0.231	0.265	0.265	0.000	0.052
12	S2	1	1	NEW	1.9	4	92.0	47.9	1.0	1.0	47.9	47.9	18.0	8.0	0.0	0.196	0.196	0.283	0.167	0.000
13	S2	0	1	MID	2.0	5	27.7	14.1	0.5	1.0	7.1	14.1	6.0	1.0	1.0	0.217	0.253	0.289	0.142	0.071
14	S2	1	0	OLD	16.4	5	230.5	14.1	1.0	18.0	14.1	252.9	22.0	5.0	38.0	0.095	0.260	0.282	0.356	0.150
15	S2	1	0	MID	7.0	5	165.8	23.9	0.0	4.0	0.0	95.4	21.0	0.0	8.0	0.127	0.175	0.175	0.000	0.084
17	S2	0	1	OLD	0.7	5	17.5	25.4	0.5	0.0	12.7	0.0	8.0	1.0	0.0	0.456	0.456	0.513	0.079	0.000
18	S2	0	1	NEW	3.6	4	46.8	12.9	0.0	3.0	0.0	38.8	10.0	0.0	12.0	0.214	0.470	0.470	0.000	0.310
19	S2	1	0	MID	3.0	5	39.5	13.2	1.0	1.5	13.2	19.8	3.0	4.0	4.0	0.076	0.177	0.278	0.304	0.202
20	S2	0	1	NEW	3.7	3	30.1	8.2	0.0	2.0	0.0	16.4	13.0	0.0	0.0	0.432	0.432	0.432	0.000	0.000
21	S2	1	0	NEW	4.7	1	20.9	4.5	0.0	5.5	0.0	24.5	2.0	0.0	3.0	0.096	0.239	0.239	0.000	0.122
22	S2	1	0	MID	3.1	5	60.3	19.6	0.0	0.5	0.0	9.8	6.0	0.0	0.0	0.100	0.100	0.100	0.000	0.000
23	S2	0	1	NEW	7.9	1	35.4	4.5	3.0	6.0	13.4	26.9	0.0	1.0	0.0	0.000	0.000	0.028	0.074	0.000
24	WS	0	1	NEW	5.2	2	49.4	9.6	1.0	7.0	9.6	67.1	1.0	1.0	11.0	0.020	0.243	0.263	0.104	0.164
25	S2	1	1	MID	4.0	5	52.8	13.2	0.5	5.0	6.6	66.2	3.0	3.0	2.0	0.057	0.095	0.151	0.453	0.030
27	S2	1	1	MID	6.1	5	149.0	24.3	2.0	2.0	48.6	48.6	10.0	9.5	11.0	0.067	0.141	0.205	0.195	0.226
28	S2	0	1	OLD	3.1	5	41.6	13.6	0.0	4.0	0.0	54.4	12.0	0.0	5.0	0.288	0.408	0.408	0.000	0.092
29	S2	1	1	MID	3.1	5	79.4	25.5	0.0	2.0	0.0	50.9	5.0	0.0	3.0	0.063	0.101	0.101	0.000	0.059
30	S2	1	1	NEW	2.7	4	23.4	8.4	0.0	2.5	0.0	21.1	2.0	0.0	3.0	0.085	0.213	0.213	0.000	0.142
31	WS	0	0	NEW	2.9	4	52.0	18.3	0.5	1.5	9.1	27.4	6.0	0.0	14.0	0.115	0.385	0.385	0.000	0.511
32	S2	1	1	NEW	1.7	4	25.8	15.3	1.0	0.0	15.3	0.0	5.0	4.0	0.0	0.194	0.194	0.348	0.262	0.000
33	S2	1	1	NEW	4.6	4	64.6	14.1	2.0	0.0	28.3	0.0	10.0	8.5	0.0	0.155	0.155	0.287	0.301	0.000
34	S2	1	1	MID	2.1	5	30.5	14.3	0.0	2.0	0.0	28.5	1.0	0.0	1.0	0.033	0.066	0.066	0.000	0.035
35	WS	0	1	MID	8.5	5	163.7	19.2	3.5	2.0	67.2	38.4	17.0	11.0	2.0	0.104	0.116	0.183	0.164	0.052
36	S2	1	0	MID	5.7	5	51.8	9.1	0.0	5.0	0.0	45.3	7.0	0.0	2.0	0.135	0.174	0.174	0.000	0.044
37	S2	1	0	MID	6.7	5	83.9	10.1	2.5	4.5	25.3	45.5	8.0	18.0	14.5	0.095	0.268	0.483	0.713	0.319

Scheme	Width	Strip	Kerb	Age band	Length km	Acc yrs	Total MVkm	Total Mveh	No of major jncs	No of minor jncs	MjJnc Mveh	MnJnc Mveh	Number of accidents			Accident rates per MVkm			Juncn rates per Mveh	
													Link	MjJnc	MnJnc	Link	MjLink	Sch	MjJnc	MnJnc
38	S2	1	0	MID	4.3	5	63.6	14.7	0.0	1.0	0.0	14.7	10.0	0.0	4.0	0.157	0.220	0.220	0.000	0.272
39	S2	1	0	OLD	1.1	5	11.5	10.5	0.0	2.5	0.0	26.2	2.0	0.0	1.0	0.173	0.260	0.260	0.000	0.038
40	S2	1	1	MID	3.0	5	43.3	14.7	0.0	2.0	0.0	29.3	11.0	0.0	10.5	0.254	0.497	0.497	0.000	0.358
41	S2	1	0	MID	9.6	5	100.5	10.5	0.0	4.5	0.0	47.2	35.0	0.0	3.5	0.348	0.383	0.383	0.000	0.074
43	S2	0	1	OLD	7.5	5	68.6	9.1	0.0	11.5	0.0	105.1	10.0	0.0	11.0	0.146	0.306	0.306	0.000	0.105
45	S2	1	0	NEW	5.0	3	45.8	9.2	1.5	0.0	13.8	0.0	10.0	2.0	0.0	0.218	0.218	0.262	0.145	0.000
46	S2	1	0	MID	2.7	5	45.3	16.5	0.5	2.0	8.3	33.1	9.0	2.0	4.0	0.199	0.287	0.331	0.242	0.121
47	S2	1	0	OLD	1.1	5	25.5	22.6	0.0	2.0	0.0	45.1	8.0	0.0	2.0	0.314	0.392	0.392	0.000	0.044
49	S2	1	0	MID	2.8	5	71.1	25.1	0.0	1.0	0.0	25.1	5.0	0.0	2.0	0.070	0.098	0.098	0.000	0.080
50	S2	1	0	NEW	2.5	3	29.6	12.1	0.5	1.0	6.0	12.1	4.0	0.5	4.0	0.135	0.270	0.287	0.083	0.331
51	S2	1	0	OLD	7.6	5	142.9	14.8	1.0	6.0	14.8	88.7	9.0	2.0	12.0	0.063	0.147	0.161	0.135	0.135
52	S2	1	0	MID	1.5	5	20.2	13.5	0.0	1.0	0.0	13.5	0.0	0.0	1.5	0.000	0.074	0.074	0.000	0.111
53	S2	1	1	OLD	5.3	5	65.4	12.4	0.0	3.5	0.0	43.4	7.0	0.0	5.0	0.107	0.183	0.183	0.000	0.115
54	S2	1	1	OLD	16.8	5	310.0	18.4	0.0	9.5	0.0	175.2	33.0	0.0	38.0	0.106	0.229	0.229	0.000	0.217
55	S2	1	1	OLD	10.0	5	135.4	13.5	0.0	9.5	0.0	128.1	21.0	0.0	12.5	0.155	0.247	0.247	0.000	0.098
56	S2	1	0	OLD	12.7	5	249.4	19.6	1.0	6.5	19.6	127.7	30.0	3.0	46.0	0.120	0.305	0.317	0.153	0.360
57	S2	1	0	NEW	2.1	1	8.6	4.1	1.0	2.0	4.1	8.2	0.0	2.0	0.0	0.000	0.000	0.234	0.490	0.000
58	S2	1	0	MID	2.8	5	66.3	24.0	0.5	2.0	12.0	48.0	3.0	0.0	4.0	0.045	0.106	0.106	0.000	0.083
59	S2	0	0	OLD	1.2	5	34.3	27.9	0.0	0.0	0.0	0.0	10.0	0.0	0.0	0.292	0.292	0.292	0.000	0.000
60	S2	0	0	OLD	3.0	5	84.7	27.9	0.0	0.0	0.0	0.0	12.0	0.0	0.0	0.142	0.142	0.142	0.000	0.000
61	S2	0	0	OLD	3.0	5	97.3	32.9	0.5	0.0	16.4	0.0	8.0	3.5	4.0	0.082	0.123	0.159	0.213	0.000
62	S2	0	1	MID	3.0	5	88.1	29.2	0.0	0.5	0.0	14.6	15.0	2.5	2.0	0.170	0.193	0.221	0.000	0.137
64	S2	1	0	MID	6.2	5	29.2	4.7	0.0	6.5	0.0	30.6	4.0	0.0	6.5	0.137	0.360	0.360	0.000	0.213
65	S2	1	1	OLD	4.3	5	29.4	6.8	1.0	2.5	6.8	17.1	4.0	1.5	2.0	0.136	0.204	0.255	0.219	0.117
66	WS	1	0	MID	9.9	5	224.0	22.7	2.0	0.0	45.4	0.0	21.0	9.0	1.0	0.094	0.098	0.138	0.198	0.000
67	S2	1	0	NEW	12.8	1	82.3	6.5	2.0	1.0	12.9	6.5	0.0	2.0	0.0	0.000	0.000	0.024	0.155	0.000
68	S2	1	1	NEW	7.8	3	66.5	8.6	2.0	4.5	17.1	38.5	3.0	4.0	3.0	0.045	0.090	0.150	0.234	0.078
69	S2	1	0	NEW	25.1	2	107.8	4.3	1.0	11.0	4.3	47.3	19.0	0.0	0.0	0.176	0.176	0.176	0.000	0.000
70	S2	1	0	NEW	18.9	1	46.6	2.5	2.0	4.0	4.9	9.9	1.0	0.0	5.0	0.021	0.129	0.129	0.000	0.506
71	S2	1	0	NEW	2.9	3	63.5	21.6	0.0	1.5	0.0	32.4	7.0	0.5	3.0	0.110	0.157	0.165	0.000	0.093
72	S2	0	0	MID	5.4	4	111.8	20.8	2.5	0.0	52.0	0.0	13.0	9.0	0.0	0.116	0.116	0.197	0.173	0.000
73	S2	1	1	MID	1.4	4	20.8	14.5	0.5	0.0	7.3	0.0	2.0	0.0	0.0	0.096	0.096	0.096	0.000	0.000
74	S2	1	0	NEW	2.9	1	10.5	3.6	1.0	2.0	3.6	7.2	0.0	3.0	2.0	0.000	0.190	0.474	0.836	0.279
75	S2	1	0	NEW	5.8	1	29.6	5.1	2.0	1.0	10.1	5.1	0.0	2.5	0.0	0.000	0.000	0.085	0.247	0.000
76	S2	1	1	MID	2.0	5	63.9	31.8	0.0	1.0	0.0	31.8	6.0	0.0	5.0	0.094	0.172	0.172	0.000	0.157
77	S2	0	1	MID	6.5	5	129.0	19.9	1.0	3.0	19.9	59.8	21.0	15.0	4.5	0.163	0.198	0.314	0.752	0.075

Scheme	Width	Strip	Kerb	Age band	Length km	Acc yrs	Total MVkm	Total Mveh	No of major jncs	No of minor jncs	MjJnc Mveh	MnJnc Mveh	Number of accidents			Accident rates per MVkm			Juncn rates per Mveh	
													Link	MjJnc	MnJnc	Link	MjLink	Sch	MjJnc	MnJnc
80	S2	0	1	OLD	0.9	5	24.7	28.8	1.0	3.0	28.8	86.3	2.0	0.0	15.0	0.081	0.687	0.687	0.000	0.174
81	WS	1	0	NEW	13.4	2	133.7	10.0	1.5	1.0	15.0	10.0	12.0	3.0	3.0	0.090	0.112	0.135	0.200	0.300
82	S2	1	0	NEW	0.6	1	1.8	3.3	0.0	0.0	0.0	0.0	0.0	0.5	0.0	0.000	0.000	0.275	0.000	0.000
84	S2	1	0	NEW	1.7	4	24.8	14.7	0.5	1.5	7.3	22.0	2.0	0.0	0.0	0.081	0.081	0.081	0.000	0.000
85	S2	1	0	NEW	2.3	1	9.3	4.1	0.0	2.0	0.0	8.3	1.0	0.0	0.0	0.108	0.108	0.108	0.000	0.000
87	S2	0	1	OLD	2.2	5	47.2	15.4	0.0	3.0	0.0	46.3	9.0	0.0	4.0	0.191	0.275	0.275	0.000	0.086
89	WS	1	0	NEW	6.1	3	57.1	9.4	4.0	1.0	37.6	9.4	2.0	1.5	0.0	0.035	0.035	0.061	0.040	0.000
90	S2	0	0	NEW	2.2	1	8.9	4.1	0.0	1.5	0.0	6.2	1.0	0.0	1.0	0.112	0.225	0.225	0.000	0.162
91	S2	0	1	MID	3.4	5	34.0	10.0	0.0	2.5	0.0	25.1	7.0	0.0	4.0	0.206	0.324	0.325	0.000	0.160
92	S2	1	0	NEW	6.1	2	29.3	4.8	2.0	2.0	9.6	9.6	2.0	1.5	0.0	0.068	0.068	0.120	0.156	0.000
93	S2	1	0	MID	7.7	5	102.7	13.3	2.0	1.5	26.6	20.0	4.0	4.0	0.0	0.039	0.039	0.078	0.150	0.000
94	WS	0	1	MID	1.4	5	16.1	9.1	0.0	2.0	0.0	18.2	2.0	0.0	2.0	0.125	0.249	0.249	0.000	0.110
95	S2	1	0	OLD	8.1	5	81.6	10.0	2.0	3.5	20.0	35.1	7.0	5.0	6.0	0.086	0.159	0.221	0.250	0.171
96	S2	0	1	NEW	1.4	2	4.8	3.5	0.0	1.0	0.0	3.5	1.0	0.0	0.0	0.207	0.207	0.207	0.000	0.000
97	WS	1	0	NEW	6.4	4	86.7	13.6	0.5	2.5	6.8	34.0	10.0	1.0	11.0	0.115	0.242	0.254	0.147	0.323
98	WS	1	0	NEW	10.6	4	154.5	14.6	3.5	7.5	50.9	109.1	15.0	13.5	7.0	0.097	0.142	0.230	0.265	0.064
100	S2	1	1	OLD	4.6	5	70.0	15.4	0.0	0.0	0.0	0.0	8.0	0.0	2.0	0.114	0.143	0.143	0.000	0.000
101	S2	1	1	OLD	2.9	5	53.1	18.2	0.0	0.0	0.0	0.0	11.0	1.5	0.0	0.207	0.207	0.235	0.000	0.000
102	S2	0	0	NEW	7.1	3	80.0	11.3	0.5	7.5	5.6	84.5	7.0	0.0	13.0	0.088	0.250	0.250	0.000	0.154
104	S2	1	1	MID	3.9	4	39.6	10.1	0.5	0.0	5.1	0.0	12.0	1.5	0.0	0.303	0.303	0.341	0.296	0.000
106	S2	1	1	NEW	5.5	2	25.8	4.7	0.5	6.0	2.3	28.0	6.0	1.0	3.0	0.232	0.349	0.387	0.429	0.107
107	S2	1	1	MID	1.0	5	21.4	22.5	0.0	0.0	0.0	0.0	4.0	0.0	0.0	0.187	0.187	0.187	0.000	0.000
108	S2	1	0	MID	2.8	5	49.9	17.9	1.5	0.0	26.8	0.0	1.0	9.5	0.0	0.020	0.020	0.211	0.354	0.000
109	S2	1	1	NEW	7.2	2	62.4	5.6	1.5	5.0	8.4	27.9	11.0	1.0	5.0	0.176	0.256	0.272	0.120	0.180
110	S2	1	0	MID	1.9	5	36.9	17.4	1.5	1.0	26.1	17.4	3.0	5.5	0.0	0.081	0.081	0.230	0.211	0.000
163	S2	0	0	NEW	1.2	2	17.9	13.0	0.5	2.0	6.5	26.1	0.0	0.0	0.0	0.000	0.000	0.000	0.000	0.000
203	WS	1	0	MID	1.1	4	13.5	12.2	1.0	0.0	12.2	0.0	3.0	6.0	0.0	0.222	0.222	0.667	0.493	0.000
205	WS	0	1	NEW	1.2	2	14.9	12.6	1.0	0.0	12.6	0.0	1.0	0.0	0.0	0.067	0.067	0.067	0.000	0.000
209	WS	1	0	NEW	1.9	3	27.1	14.6	1.0	0.0	14.6	0.0	2.0	0.5	0.0	0.074	0.074	0.092	0.034	0.000
244	WS	0	1	NEW	1.2	4	13.9	11.8	0.5	0.5	5.9	5.9	1.0	0.5	0.0	0.072	0.072	0.108	0.085	0.000
248	WS	0	1	OLD	1.6	5	37.0	23.9	0.0	2.5	0.0	59.7	7.0	0.0	1.0	0.189	0.216	0.216	0.000	0.017
286	WS	1	0	NEW	2.0	2	18.6	9.2	1.0	0.0	9.2	0.0	1.0	0.5	0.0	0.054	0.054	0.081	0.054	0.000
303	S2	1	1	MID	4.7	4	59.3	12.6	2.0	2.0	25.2	25.2	10.0	8.0	11.5	0.169	0.363	0.498	0.317	0.456
305	S2	0	1	NEW	2.8	2	34.8	12.6	0.5	1.0	6.3	12.6	0.0	0.0	4.0	0.000	0.115	0.115	0.000	0.319
309	S2	1	0	NEW	2.2	3	18.8	8.7	0.5	3.0	4.3	26.0	2.0	0.5	1.0	0.106	0.160	0.186	0.115	0.038
344	S2	0	0	NEW	2.5	4	29.8	12.1	1.0	2.0	12.1	24.2	6.0	3.0	4.0	0.201	0.336	0.436	0.248	0.165

Scheme	Width	Strip	Kerb	Age band	Length km	Acc yrs	Total MVkm	Total Mveh	No of major jncs	No of minor jncs	MjJnc Mveh	MnJnc Mveh	*Number of accidents* Link	MjJnc	MnJnc	*Accident rates per MVkm* Link	MjLink	Sch	*Juncn rates per Mveh* MjJnc	MnJnc
348	S2	1	0	OLD	2.2	5	59.9	24.1	0.0	3.5	0.0	84.2	7.0	0.0	2.0	0.117	0.150	0.150	0.000	0.024
386	S2	1	0	NEW	7.6	2	62.3	8.2	2.0	0.0	16.3	0.0	9.0	0.0	0.0	0.144	0.144	0.144	0.000	0.000
Totals					531.3	425	7090.1	1593.5	84.5	290.5	1105.3	3671.4	889.0	239.5	511.0	0.125	0.197	0.231	0.217	0.139
108 Schemes														Minimum		0.000	0.000	0.000	0.000	0.000
														10 percentile		0.020	0.066	0.085	0.000	0.000
														Lower quartile		0.073	0.108	0.143	0.068	0.035
														Median		0.111	0.191	0.227	0.165	0.105
														Upper quartile		0.182	0.260	0.288	0.281	0.174
														90 percentile		0.222	0.360	0.408	0.453	0.319
														Maximum		0.456	0.687	0.687	0.881	0.511

Appendix 5 shows the following data:

Width = Carriageway width, given as S2 (nominal width 7.3m) or WS2 (10m)

Strip = 1 if the scheme has hardstrips over most of its length, 0 otherwise

Kerb = 1 if the scheme has kerbs over most of its length, 0 otherwise

Age band of the scheme, classified as Old (opened in or before 1980), Mid (opened 1981-1985), or New (opened 1986-1990)

Length in km

Acc Yrs = number of years for which accident data are available

Total vehicle-km, used for calculation of link accident rates

Total vehicles throughput = AADT summed over Acc Yrs

Number of major and minor junctions on the scheme, adjusted for scheme ends and junctions with more important roads, see text

Major and minor junction vehicle throughput = the number of junctions times the total vehicle throughput, for calculation of junction accident rates

Number of accidents on links and at major and minor junctions

Accident rates, per million vehicle-km, for links only, major links and schemes, as described in the text

Accident rates, per million vehicles passing through the junctions, for major and minor junctions.

Abstract

This study of single-carriageway trunk roads of modern design quantifies the relationships between the numbers of injury accidents that occur, and the traffic and road layout variables that determine them.

The study covers 103 new schemes, opened since 1968, on non-built-up single-carriageway trunk roads in England. The techniques of generalised linear modelling were used to develop predictive relationships between numbers of accidents, traffic flow and geometric features of the road. The main results of this study refer to road links, that is, the stretches of road between junctions.

The study was commissioned by the Traffic, Safety and Environment Division of the Highways Agency and was undertaken by the Transport Research Laboratory.

Related publications

TRL335 *Accidents on modern rural dual-carriageway trunk roads* by D A Walmsley, I Summersgill and A Payne. 1998 (price £50, code L)

TRL334 *The relationship between road layout and accidents on modern rural trunk roads* by D A Walmsley and I Summersgill. 1998 (price £35, code J)

TRL281 *Accidents at urban mini roundabouts* by J Kennedy. 1998 (price £50, code P)

TRL135 *Accidents at 3-arm traffic signals on urban single-carriageway roads* by M C Taylor, R D Hall and K Chatterjee. 1996 (price £50, code N)

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