TRANSPORT RESEARCH LABORATORY

# Accidents on modern rural dual-carriageway trunk roads

## Prepared for Traffic, Safety and Environment Division, Highways Agency

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## Executive Summary

In 1991 there was 2,528 km of dual-carriageway trunk road in England. 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 dual-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 112 new schemes, totalling 622 km in length, which comprised about half of the new non-built-up dual-carriageway trunk road schemes in England which opened between 1968 and 1989. The accident period studied was from 1979 to 1992. There were 9097 accidents during this period on the schemes studied, of which 5712 were link accidents, 1278 occurred at major junctions and 2107 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 curvature, gradient, and the presence of kerbs and safety fences. 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 the presence of kerbs, hardstrips and safety fences.

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

- Modern dual-carriageway trunk roads are safer than traditional A-roads, having about 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. This is due to improvements in road design, in particular the more widespread use of hardstrips, safety fences and other road design features which contribute to fewer accidents, and to fewer and better-designed junctions.
- In addition to this improvement due to road design, modern roads are getting safer all the time due to improvements in vehicle design and driver behaviour.
- Roads with hardstrips or continuous edge lining are safer than those with kerbs at the carriageway edge, having about 23 per cent fewer accidents, all else being equal.
- Roads with median safety fences and fewer and better-designed junctions have fewer accidents, all else being equal.
- Bends, hills, right-turn accesses and obstructions on the median all lead to a slightly increased accident risk, all else being equal.

# 1 Introduction

## 1.1 Background

In 1991 there was 2,528 km of dual-carriageway trunk road in England (Department of Transport, 1992). 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, DETR) 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 dual-carriageway trunk roads of modern design. The study was restricted to non-built-up dual-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 single-carriageway roads (Walmsley, Summersgill and Binch, 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 dual-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 about half of the modern rural dual-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 dual-carriageway roads, both all-purpose and motorway 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 dual-carriageway rural trunk roads, 271 dual-carriageway trunk road schemes which opened between 1968 and 1989 were examined, and a sample of 120 schemes was selected for study as resources did not permit the inclusion of all 271. It was necessary to eliminate a number of these due to non-availability of data, leaving 112 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 (including grade-separated 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 554 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 1992. These data were supplied to an external contractor who performed two tasks. Firstly, they located each accident on a map and assigned it to the appropriate minor link or junction. Secondly, they categorised each accident according to 7 or 8 key variables so that they could later be allocated to an accident type. There were 9097 accidents in the data set, of which 5704 were link accidents and 3368 junction accidents, plus 25 undefined accidents. 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 1992, but no data were available for some census points; these gaps were filled by requesting data from 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 10m 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. Details are 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. 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. Details are presented in Section 9.

Two types of extra variables were derived for testing in the models. One was a number 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). The other was an assessment of the overall quality of the road. Details are presented 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 statistical package (Alvey *et al*, 1977) 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 Sections 15 (dealing with 2-lane dual carriageways) and 16 (dealing with 3-lane dual carriageways).

A summary of the model results and their implications for the existing standards are discussed in Section 17, and the main conclusions from the study are summarised in Section 18.

## 2 Road 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.

Although Highway Link Design applies to both single and dual carriageways, it mainly addresses two problems with earlier designs of single-carriageway roads, in particular the previous design known as Layout of Roads in Rural Areas (LRRA) (Department of the Environment, 1968). 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.

One of the main features of HLD is that it maximises overtaking opportunities on single-carriageway roads by including as many straight, or nearly straight, stretches as possible. Where the road bends, the road is made sufficiently curved so that drivers can clearly perceive that overtaking is not possible; intermediate radius curves, where drivers might be tempted to overtake where it is not safe to do so, are avoided.

As far as the design of all-purpose dual carriageways is concerned, Highway Link Design did not introduce radical changes. The main criterion was to permit light vehicles to maintain the design speed. Unlike single carriageways, there was no limitation on the use of horizontal or vertical curves, apart from an absolute minimum value for the radius, and the co-ordination of design elements mainly involved the design and optimisation of aesthetic alignments.

HLD specifies design speeds for new all-purpose roads in a series of steps, namely 50, 60, 70, 85, 100 and 120 km/h. 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.

## 3 Literature survey

A review of the literature was made to identify variables that had been tested in previous studies of accidents on dual-carriageway rural trunk roads, both all-purpose and motorway, and to see which variables had been shown to have an effect on accidents. There were very few relevant studies. Simpson and Brown (1988) have reviewed three unpublished studies of accidents on all-purpose dual-carriageway roads that were conducted under contract to the Department of Transport in the 1980s. These, and some other relevant studies, are described below.

### 3.1 Study of single/dual-carriageway interfaces

The first study reviewed by Simpson and Brown (1988) concerned safety on highways being built or improved as a mixture of single and dual carriageways along a route. There was concern that such changes of carriageway type might cause an increase in accident risk on single-carriageway sections which followed a dual-carriageway section. In order to investigate whether this might be so, DOT commissioned a separate study. The aim was to derive accident rates for lengths of single-carriageway roads adjacent to and downstream of an interface with a dual-carriageway section and to determine whether these rates were significantly higher than those for otherwise similar single-carriageway roads.

The sites chosen were rural trunk or principal roads. Where the interface was a junction, the single carriageway was to be straight ahead with the same road number. A total of 86 sites was selected in England, of which the single-carriageway components comprised carriageways with nominal widths of 7.3m, 7.3m with hardstrip, and 10m. Total accident rates on three successive individual kilometre lengths from the interface on the downstream single-carriageway side were not significantly different from rates on ordinary lengths of single carriageway.

### 3.2 Study of safety fences

The second was a study of the effectiveness of safety fences (crash barriers) in reducing accidents and the establishment of criteria for their installation. Whilst safety fence installation has been standard on motorways in England since the early 1970s, and 98 per cent of motorways have them, only about 10 per cent of dual carriageways were so equipped at the end of 1986. The study was commissioned by DOT and conducted under contract to TRL. Accidents on 150 km of fenced and 150 km of unfenced two-lane rural all-purpose dual-carriageway roads in England were compared over the period from 1979 to 1984 inclusive. The techniques of generalised linear modelling were used in the regression analysis, and 15 variables and factors were tested, as follows:

- total vehicle flow;
- presence of a safety fence;
- percentage of goods vehicles;
- presence of mid-lane markings;
- presence of cats-eyes between lanes;
- presence of cats-eyes on offside edge;
- presence of offside kerb;
- presence of nearside hardstrip;
- median width;
- gradient;
- superelevation;
- minimum curve radius;
- presence of road lighting;
- presence of offside edge markings;
- presence of offside metre strips.

Accidents were divided into median (893) and non-median (2473) accidents. The presence of a safety fence was associated with a reduction of 28 per cent in median accidents, which was statistically significant at the 1 per cent level. The review indicates that the presence of a safety fence was associated with lower accident frequencies for total accidents, but does not indicate whether this was statistically significant.

Sections of road with an offside kerb, which tended to be the older types of dual carriageway, had an accident rate, for both median and total accidents, which was about double that of sections without a kerb. The terms representing the presence of offside cats-eyes and offside metre strips were also present in a model for total accidents, but neither the magnitude of the effect on accidents nor the level of statistical significance of these variables was stated. Vehicle flow also appeared in this model.

### 3.3 Study of accident rates on motorways and dual carriageways

The third study reviewed by Simpson and Brown (1988) was a study of accident rates on motorways and dual carriageways commissioned by DOT. Only the part of the study that related to all-purpose dual carriageways is of direct concern here.

The study was conducted on 85 sites in England, comprising 450 km of rural trunk and principal two-lane dual carriageway in lengths greater than 5 km which were not lit, did not have central safety fences and had no frontage development. Data on non-junction injury accidents for the period 1980 to 1983 inclusive were used in the analysis, which was based on tabulations.

The site characteristics were stratified into bands to test whether any had a statistically significant effect on accident rates. The characteristics tested were:

- total vehicle flow;
- percentage of goods vehicles;
- the ratio of flow to capacity (capacity taken as 45000 vehicles/day for 2-lane dual carriageway all-purpose roads);
- the length between junctions;
- the type of edge treatment (metre strips or not).

Of these, only total vehicle flow and the edge treatment had a statistically significant relationship with accidents. The relationship with edge treatment may be related to the age of the road, since only the older roads did not have metre strips.

Table 3.1 shows non-junction accident rates for these all-purpose two-lane rural dual carriageways and motorways. Those all-purpose dual-carriageway roads that were modern at the time of the study (ie those built between 1973 and 1980) were separated from those built in the 1960s. All of the latter had kerbs rather than metre strips. An average non-junction accident rate of 11 injury accidents per 100 million vehicle-km occurred on the more modern carriageways, but the rate was 15 injury accidents per 100 million vehicle-km on the older carriageways. The analysis of motorway accidents in the same study showed

**Table 3.1. Non-junction accident rates for all-purpose dual carriageways and motorways (1980-1983)**

| *Road type* | *Number of Sites* | *Length km* | *Traffic MVkm* | *Personal injury accidents* | *Accident rate accs per MVkm* | *% fatal or serious* |
|---|---|---|---|---|---|---|
| Modern 2-lane all-purpose dual carriageway built 1973-80 | 85 | 397 | 8688 | 947 | 0.110 | 46 |
| Selected less modern sections | 11 | 61 | 1526 | 229 | 0.150 | 40 |
| 2-lane motorway | 40 | 195 | 7134 | 804 | 0.113 | 30 |
| 3-lane motorway | 164 | 1076 | 56247 | 5591 | 0.099 | 32 |

*(Simpson and Brown 1988)*

that non-junction accidents on two-lane motorways occurred at a rate of 11 injury accidents per 100 million vehicle-km. Thus the non-junction injury accident rate was the same for modern all-purpose dual carriageways as for motorways. The percentage of fatal and serious accidents was greater (46 per cent) on the all-purpose roads than on the motorways (30 per cent).

### 3.4 Other studies of dual-carriageway accidents

Shrewsbury and Sumner (1980) studied 100m sections of several types of road including all-purpose dual carriageways. Accident rate was affected by horizontal radius, gradient and sight distance. The rate increased sharply for tight curves with radii less than 400m and on downhill gradients, but much less so for corresponding uphill gradients. Sight distances of less than 200m were associated with higher accident rates.

Turner and Thomas (1986) studied accidents on dual-2 and dual-3 lane motorways in Britain. They showed that accidents were related to traffic flow and that the accident rate was slightly greater for dual 2-lane motorways (0.103 injury accidents per MVkm) than for dual 3-lane motorways (0.099 injury accidents per MVkm). Geometric variables were not tested.

Hall and Surl (1981), working under contract to TRL, published preliminary findings of a study of accidents at 4-arm roundabouts, traffic signals and priority junctions on all-purpose dual carriageways. All the junctions were at-grade. The results showed that the flow of vehicles by individual turning movements affected the number of injury accidents. No feature or geometric variables were tested.

Maycock and Hall (1984) developed the analysis of roundabouts further to include the testing of feature and geometric variables. These showed that the numbers of injury accidents was affected by:

- entering and circulating flows;
- the percentage of motor-cycles;
- entry path curvature;
- entry width;
- approach width;
- approach curvature;
- the angle between the arms;
- central island diameter;
- inscribed circle diameter.

### 3.5 Studies of single-carriageway roads

Unlike dual carriageways, single-carriageway rural roads have been the subject of a large number of accident studies. The results of these have been reviewed by Walmsley, Summersgill and Binch (1998).

## 4 Scheme selection

### 4.1 Identification of modern dual-carriageway schemes

The objective of the study was to investigate accidents on rural dual-carriageway roads which have been designed and constructed in recent years, that is, under the LRRA or HLD design standards. It was therefore decided that the initial selection of schemes would include all dual-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 all-purpose dual-carriageway trunk 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). The lists for earlier years (1968-75) did not distinguish between single- and dual-carriageway roads, so examination of maps and comparison with the list of sites selected for the single-carriageway study was necessary.

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. This resulted in a definitive sampling frame of 271 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.

### 4.2 Sample selection

It was beyond the resources of the study to analyse the full set of 271 schemes which had been identified from White Papers. A simple random sample of 120 schemes was therefore drawn from the full set of 271. A few of the sampled schemes were found to be unsuitable because of missing data, and were replaced by further schemes from a reserve list. The distribution of the sample by region, length and opening date was shown to be not statistically significant (at the 5 per cent level) from the full set.

During the video viewing by the contractors, some further schemes were found unsuitable and these were eliminated, without replacement, from the sample. This left 112 schemes in the final sample; they are listed in Appendix 1, together with their lengths and opening dates.

### 4.3 Identification of links and junctions

This study was mainly concerned with accidents on links, that is, stretches of road between junctions. In the dual-carriageway study it was found that the identification of junctions, and therefore of links, was not as simple as in the single-carriageway study, because on dual carriageways junctions tend to be more complex than on single carriageways. The intersection of two roads, one or both of which is a dual carriageway, can comprise a series of slip roads, acceleration and deceleration lanes, cross-overs and islands.

It was decided that the whole intersection between two roads should be regarded as the junction, rather than every slip road being regarded as a junction, because this enabled accident rates to be calculated for the whole intersection, which is likely to be of greatest interest to the road designer.

A junction was therefore defined as the stretch of road between the first point at which a slip road or deceleration lane diverges from the main carriageway to the last point at which a slip road or acceleration lane meets it. In some cases, where two junctions were close together and the arrangement of joint lane markings and slip roads overlapped, the whole complex was defined as a single junction. This definition implied a wide variety of designs of junctions, with a separate junction type being required for nearly every individual junction. It also meant that a junction could extend over a considerable distance, and might include an ‘internal link’, of main carriageway between first exit and last entrance, on which some accidents might occur.

Because of the difficulty of seeing on video the structure of the junction away from the main carriageway, and the wide variety of possible structures, only a limited range of information about the junction was coded, consisting essentially of the length from start to end, the number of entry and exit point on the main carriageway, and details of road markings.

On dual carriageways, the location and structure of a junction (alignment of slip roads etc) is commonly different on the two carriageways. Indeed, there may be a junction on one carriageway but not on the other. The link and junction structure for each scheme was therefore identified separately for the two carriageways.

### 4.4 Definition of units for study

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. A major junction extends across both carriageways;

***Minor junction:*** any other properly-marked junction where traffic on the major road has priority. A minor junction is located on one carriageway only, and may or may not be linked by a cross-over to a minor junction on the opposite carriageway. Grade-separated junctions are minor junctions, because traffic on the scheme road does not have to give way;

***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.

***Internal link:*** that part of the main carriageway of the scheme road which lies within a junction, that is, between the first off-slip and the last on-slip. An internal link is regarded as being part of the junction, not part of any minor link.

***Junction length:*** the length of the internal link of a junction, or the total length of the internal links of junctions on a scheme.

***Link length:*** the length of a minor link, or the total length of the minor links on a scheme, not including the length of any junction.

***Major link length:*** the length of a major link, or the total length of the major links on a scheme, including the length of any minor junctions which the major link may contain, but not including the length of any major junctions.

***Scheme length:*** the total length of a scheme, including the length of its links and junctions.

***Link accidents:*** accidents which occur on the scheme road but not within a junction.

***Junction accidents:*** accidents which occur on the scheme road at a junction or on any of the slip roads or internal links associated with a junction.

Some accidents which STATS19 classes as link accidents would be classed as junction accidents according to these definitions (accidents on internal links or slip roads, for example), and vice versa (access accidents, for example).

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 1992.

For the purpose of extracting accident data, the 120 schemes in the definitive list were specified by an Ordnance Survey grid reference box, drawn to the next 1 km outwards 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 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.

### 5.2 Location of accidents

Data on 18909 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 for other reasons. The 14239 accidents which remained 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: 172 accidents which occurred on the old road which had been superseded by the scheme; 3154 accidents which occurred on the new road, but beyond the limits of the scheme under consideration (mainly due to the 1 km margin in specifying grid references); 1190 accidents which occurred on another road; 414 accidents which did not fall on any road, or had unexplained or irrational coordinates. As a result, 9309 accidents remained for analysis, of which 5837 occurred on links and 3472 at junctions.

Each remaining accident was then assigned to a direction of travel (the A or B direction), using the compass points information on the STATS19 vehicle records related to the map of the scheme. In those few cases where the vehicles involved were travelling in opposite directions, the accident was allocated to the direction in which the vehicle which caused the accident was travelling - for example, the vehicle which crossed the central reservation.

### 5.3 Quality control (Accident coding)

Care was taken throughout the accident coding process to maintain coding accuracy. An experienced technician was used to code the majority of the accidents and to check a sample of others. The contractors appointed a Quality Control Officer who was not involved in the day-to-day running of the project to carry out accuracy checks and re-brief the coders as required.

TRL also made a number of checks on the accident data after delivery. As a result, a number of corrections were made, including some to the scheme structure, which involved the deletion of some links and one whole scheme. Following these corrections, there remained 9097 accidents in the data set, of which 5712 were link accidents, 1278 occurred at major junctions and 2107 at minor junctions.

### 5.4 Allocation of accidents to groups

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

The classification scheme, which took into account the number of vehicles involved, the manoeuvres being carried out, and similar criteria, was based on the accident groups for the single-carriageway study, with some obviously necessary changes, for example the inclusion of a group for accidents caused by a vehicle crossing the central reservation.

One particular difficulty was that the occurrence of overtaking is less clearly defined on dual carriageways than on single carriageways. Many vehicles cruise for considerable periods of time in the offside (overtaking) lane, passing other vehicles as they do so, so it may not be meaningful to say whether or not they were overtaking. Vehicles also change lane, in order to overtake or for other reasons. The distinction between the STATS19 ‘overtaking’ and ‘changing lane’ manoeuvres was therefore somewhat arbitrary, so they were amalgamated into a single group.

The detailed list of group definitions, and the number of accidents in each, is shown in Appendix 2.

### 5.5 Allocation of accidents to classes

One important way in which accidents could be grouped was according to the number of vehicles involved. In the dual-carriageway study, accidents were therefore classified into accidents involving 1, 2, and 3 or more, vehicles. These groupings are referred to as *classes*. Accidents which involved pedestrians, accesses or parked vehicles were not allocated to any class.

The number of accidents in each class is shown in Appendix 3.

## 6 The selection of explanatory variables

The list of variables, both basic and derived, that were measured and assembled for the analysis are presented in Appendix 4, with a note of the average and range of values for each variable. 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, presence of kerb, presence of hardstrip and presence of central safety fence.

Secondly, a review of the available literature was made to see what variables had been used in similar studies in the past. Unfortunately, there were few relevant studies of dual carriageways or motorways. A summary of findings was given in Section 3. Where appropriate, the variables and factors which were measured in the corresponding study of rural single-carriageway trunk roads (Walmsley, Summersgill and Binch, 1998) were also included, with modifications for the two carriageways as necessary.

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 dual-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 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 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 1992 (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 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).

### 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 1992, with a standard error of 0.13 per cent, giving a 95 per cent confidence interval from 4.2 to 4.7 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.

For the purposes of generalised linear modelling, it was decided to deduce a traffic growth factor for each census point and each scheme individually. The reasons for this are explained in Section 13.5.

The growth rates for each census point were estimated by fitting a log-linear regression line to the available traffic data for each census point individually. The census-point-specific growth rates obtained ranged from -16.1 per cent to +20.2 per cent, with a mean of 4.8 per cent and a standard deviation of 4.2 per cent.

Scheme-specific growth rates were also estimated, by fitting a set of lines of common slope to all census points on each scheme. The scheme-specific growth rates obtained ranged from -14.3 per cent to +20.2 per cent, with a mean of 4.7 per cent and a standard deviation of 4.1 per cent.

# 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 (though the single-carriageway study did not find alignment to be significant for most accident types). They also 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, it was decided to obtain alignment data 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 10m, 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 10m 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 (10m) 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 10m point and its radius of 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 1000m equates to a curvature of 57 degrees (1 radian) per km, a radius of 500m equates to 115 degrees (2 radians) per km, and so on. Maximum curvature implies minimum radius of curvature.

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 HLD Advice Note for a design speed of 120 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).

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, safety fences 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 single-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. 228 video tapes were required to cover the schemes of interest in both directions, of which 50 were available from the single-carriageway study.

### 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). Variables for links and minor junctions were observed for each scheme in each direction separately. Major junction variables were observed from the videos for both directions.

The list of variables to be measured in the dual-carriageway study was largely the same as for the single-carriageway study, making allowances for differences in road layouts, for example the inclusion of details of safety fences and markings on the central reservation. Details to be recorded at junctions were considerably reduced, because of the complexity and variability of junction forms as discussed earlier (Section 4.3), and different schemes were used for coding major and minor junctions. The full list of variables measured from the videos is shown in Appendix 4.

The video observations of the 112 schemes identified 554 minor links, and included 13243 75-metre sections. There were 57 major junctions and 479 minor 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 Kerb, hardstrip and safety fence variables

Highway Link Design (HLD) specifies that dual-carriageway roads should be built with 1-metre hardstrips and without kerbs, except for traffic flows below about 25,000 per day, where kerbs are permitted (but where economic analysis might show that a wide single-carriageway road would suffice). 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.

Kerbs, hardstrips and median safety fences do not necessarily occupy the whole length of a minor link. Therefore, each minor link was coded according to whether each of these features occupied more or less than 50 per cent of the length.

In order to estimate this factor, the edge treatment and central reservation type codes for each 75-metre section from the video data file were re-coded to 1 if a kerb, hardstrip or median fence was present and 0 if not. The proportion of the link on which there was a kerb, hardstrip, or median fence was calculated from the number of 75-metre sections on the link where the feature was present, ignoring any stretches affected by junctions, climbing lanes etc. The presence/absence factor was then set according to whether the proportion was greater than or less than 0.5.

To enable accident rates to be analysed by scheme, rather than by minor link, a proportion and factor were similarly calculated for each scheme. The factors for presence of kerb, hardstrip and median fence for each scheme are shown as part of Appendix 5.

### 10.3 Carriageway width

During the observation of videos, carriageway width was found not to vary significantly, except insofar as dual carriageways had 2 or 3 lanes. Carriageway width was therefore coded simply as a 2-level factor D2 or D3 according to the number of lanes. No link contained a mixture of widths, so it was not necessary to estimate the predominant width for the link. 36 links were assigned to the D3 category.

To enable accident rates to be analysed by scheme, the width which occurred over the greater part of each scheme by length was estimated. Only a few schemes contained a mixture of D2 and D3 links, and these consisted mostly of short D3 links in a predominantly D2 scheme, so no difficulty was experienced in assigning schemes to one width category or the other. Five such mixed schemes (numbers 3, 7, 80, 85 and 87) were assigned to the D3 category; these schemes accounted for 29 of the 36 D3 links.

### 10.4 Derived variables

Two types of extra variables were derived from the alignment and geometric data described above for testing in the models. One was a number 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). The other was an assessment of the overall quality of the road.

### *10.4.1 Speed variables*

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 dual-carriageway roads as:

$$V = BS_9 - 4.6 - [0.1*B] + [0.25*H_-] - [0.006*F].$$

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:

$$V = BS_{10} - [0.1*B] - [0.28*H_+] - [0.006*F].$$

In these formulae, the variables have the following meanings:

$BS_9$ = Base speed in the COBA-9 formula, with the following values:

- 102 km/h for D3AP (3-lane dual-carriageway all-purpose) roads
- 98 km/h for D2AP (2-lane dual-carriageway all-purpose) roads

$BS_{10}$ = Base speed in the COBA-10 formula, with the following values:

- 115 km/h for D3AP roads
- 108 km/h for D2AP roads

$B$ = Bendiness
$H_-$ = Down-hilliness
$H_+$ = Up-hilliness
$F$ = vehicles / hour / direction

These formulae are similar to those used in the single-carriageway study, but include fewer variables.

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 $A_C = 6.6 + 0.1*B$,and $L_C$ is obtained from tables and depends on the carriageway width, the verge width and the number of accesses and junctions. For dual carriageways the Layout Constraint is 6 for D3AP roads, and 9 or 10 for D2AP roads depending on whether there are a low or medium number of accesses per kilometre.

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.

**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* | *$V_{50,wet}$ average* |
|---|---|---|---|---|---|
| 100B: | all D2 | 1 | 82.9 | 87.0 | 77.9 |
| 100A: | all D2 | 8 | 77.5 | 86.7 | 81.6 |
| 120B: | D2 | 128 | 87.6 | 97.5 | 91.0 |
| | D3 | 23 | 93.7 | 108.2 | 91.6 |
| | All | 151 | 88.5 | 99.2 | 91.1 |
| 120A: | D2 | 381 | 91.4 | 102.6 | 95.0 |
| | D3 | 13 | 95.9 | 109.6 | 93.8 |
| | All | 394 | 91.5 | 102.8 | 95.0 |
| All ranges: | D2 | 518 | 90.2 | 101.0 | 93.8 |
| | D3 | 36 | 94.5 | 108.7 | 92.4 |
| | All | 554 | 90.5 | 101.5 | 93.7 |

The average speed according to the COBA-10 formula was 101.5 km/h, with a range from 75.4 to 113.1 on individual minor links. The average speed according to the COBA-9 formula was 90.5 km/h, with a range from 73.4 to 101.9. The average difference was 11.07 km/h.

The speeds according to the three methods of calculation (COBA-9, COBA-10 and design speed), plus $V_{50\ wet}$, were tested as variables in the regression models described later, 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.

### *10.4.2 Road quality*

Dual-carriageway roads can vary greatly in overall quality, from high standard, high speed roads that to the driver are little different from motorways, to older by-passes which might have frequent roundabouts and junctions. Road quality is of course inherent in many of the design variables already described, for example, a good modern road will usually have grade-separated junctions and hardstrips, few accesses, and no sharp bends or gradients. These variables were tested individually in the models.

However, in the analysis it was found that the fit of the models was improved by including also a variable describing an overall assessment of the quality of the road. This was based mainly on objective criteria such as the ratio of major to minor junctions and the proportion of junctions with slip roads and deceleration lanes, but a subjective judgment as to the general appearance of the road was also included. Road quality was not included as a separate variable in the single-carriageway study.

Roads were assigned to one of four categories:

***Best Quality****:* Good, high-speed modern roads with an open aspect, near-motorway standard, where high speeds can be attained. Minor junctions mostly grade-separated or with long splayed slip roads, usually without cross-overs. No major junctions.

***Good Quality****:* As Best quality, but with some large roundabouts with long curved approaches.

***Lower quality****:* Older-style roads. Minor junctions often T-junctions or crossroads with only short deceleration lanes. Major junctions usually smaller roundabouts with straight approaches, and shorter distances between them.

***Urban Bypass****:* Modern roads in semi-urban areas. Good, modern roads but with too many junctions for Good quality. They may have frequent small minor junctions with deceleration lanes and few crossovers. Roundabouts may be present and can be frequent.

The assignment resulted in 57 Best, 22 Good, 19 Lower and 14 Urban Bypass quality schemes.

# 11 Descriptive analysis of accidents

This section analyses the characteristics of the accidents on the 112 dual-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.

The mean accident severity ratio was 31 per cent, which was closer to the severity for motorways than for all-purpose dual carriageways identified in earlier work (Section 3.3). Omitting those schemes with fewer than 20 accidents in total, the distribution of severities over the individual schemes had a minimum and maximum of 11 and 66 per cent respectively, similar to the schemes in the single-carriageway study. The mean accident severity was a little higher than 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 4.6 per cent, which was also similar to the national fatality ratio (4.3 per cent), but was appreciably lower than that for the schemes in the single-carriageway study (7.3 per cent).

The average accident frequency for the schemes studied was 7.0 accidents per year, with a minimum of 1.54 (A66 Cumbria, with similar values on several other schemes) and a maximum of 13.67 on a busy stretch of the A2 on the edge of London. Accident frequency takes no account of scheme length or traffic flow, so it was likely to be higher on longer and busier schemes.

The average accident density (accidents per km per year) was 0.63 accidents per year per km. The minimum and maximum were 0.15 (A361 North Devon Link) and 3.93 (A10 in Hertfordshire) respectively. Accident density takes no account of traffic flow, so it was likely to be higher on busier schemes.

### *11.1.2 Number, frequency and severity of accidents by location and junction type*

The distribution of accidents by location (link or junction) is shown in Table 11.3, and Table 11.4 compares junction types.

The accident frequencies for major and minor junction accidents were similar, at 1.0 and 1.6 accidents per year respectively, but the frequency for links was much higher at 4.4 accidents per year. These figures were scheme-based statistics and therefore depended on the length of the scheme and the number of junctions. When the number of junctions was allowed for, as in Table 11.4, accident frequencies were significantly higher for major junctions (2.2 per junction-year) than for minor junctions (0.4), with an overall average of 0.5 (similar to single carriageways).

The accident severity for link accidents was 34 per cent, with 17 per cent for major junctions and 30 per cent for minor junctions). This indicates that the greater number of accidents at major junctions (mainly roundabouts) resulted mainly in slight injuries.

### *11.1.3 Number, frequency and severity of accidents by accident group and class*

Table 11.5 presents the number of accidents classified as fatal, serious or slight, broken down by accident groups. The 19 accident groups refer to different vehicle manoeuvres and are listed in Appendix 2. Groups A to E are junction accidents and groups J to Z link accidents.

Among the link accidents, the greatest number of 1-vehicle accidents were due to the vehicle leaving the carriageway on the nearside (group M, 10 per cent of all accidents), and the greatest number of multi-vehicle accidents were due to one vehicle overtaking or changing lane (group T, 10 per cent of all accidents). Accident group Z also accounted for about 10 per cent of all

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

| *Scheme* | *Scheme length (km)* | *Scheme years* | *Number of accidents* | | | | *Accident frequency (acc/year)* | *Accident severity (% fatal & serious)* | *Accident density (acc/year per km of scheme)* |
|---|---|---|---|---|---|---|---|---|---|
| | | | *Fatal* | *Serious* | *Slight* | *Total* | | | |
| 1 | 28.36 | 12 | 13 | 42 | 81 | 136 | 11.33 | 40.4 | 0.40 |
| 3 | 25.74 | 14 | 6 | 54 | 72 | 132 | 9.43 | 45.5 | 0.37 |
| 6 | 13.92 | 14 | 6 | 29 | 78 | 113 | 8.07 | 31.0 | 0.58 |
| 7 | 10.22 | 14 | 10 | 35 | 107 | 152 | 10.86 | 29.6 | 1.06 |
| 8 | 22.87 | 10 | 4 | 26 | 63 | 93 | 9.30 | 32.3 | 0.41 |
| 9 | 13.74 | 14 | 10 | 37 | 103 | 150 | 10.71 | 31.3 | 0.78 |
| 10 | 14.32 | 14 | 1 | 33 | 87 | 121 | 8.64 | 28.1 | 0.60 |
| 11 | 7.94 | 11 | 8 | 61 | 274 | 343 | 31.18 | 20.1 | 3.93 |
| 12 | 5.94 | 10 | 1 | 16 | 96 | 113 | 11.30 | 15.0 | 1.90 |
| 13 | 6.72 | 10 | 5 | 14 | 10 | 29 | 2.90 | 65.5 | 0.43 |
| 14 | 8.06 | 6 | 0 | 18 | 39 | 57 | 9.50 | 31.6 | 1.18 |
| 15 | 13.05 | 11 | 4 | 13 | 21 | 38 | 3.45 | 44.7 | 0.26 |
| 16 | 10.90 | 8 | 2 | 19 | 49 | 70 | 8.75 | 30.0 | 0.80 |
| 17 | 5.10 | 5 | 1 | 6 | 15 | 22 | 4.40 | 31.8 | 0.86 |
| 18 | 19.58 | 13 | 9 | 57 | 64 | 130 | 10.00 | 50.8 | 0.51 |
| 19 | 14.16 | 13 | 5 | 24 | 32 | 61 | 4.69 | 47.5 | 0.33 |
| 20 | 12.32 | 10 | 5 | 12 | 24 | 41 | 4.10 | 41.5 | 0.33 |
| 21 | 4.82 | 9 | 1 | 6 | 33 | 40 | 4.44 | 17.5 | 0.92 |
| 22 | 11.28 | 14 | 5 | 31 | 97 | 133 | 9.50 | 27.1 | 0.84 |
| 23 | 3.10 | 14 | 0 | 7 | 17 | 24 | 1.71 | 29.2 | 0.55 |
| 24 | 2.26 | 12 | 2 | 16 | 27 | 45 | 3.75 | 40.0 | 1.66 |
| 26 | 8.88 | 14 | 3 | 19 | 54 | 76 | 5.43 | 28.9 | 0.61 |
| 27 | 16.27 | 14 | 8 | 34 | 137 | 179 | 12.79 | 23.5 | 0.79 |
| 28 | 12.46 | 7 | 4 | 15 | 94 | 113 | 16.14 | 16.8 | 1.30 |
| 29 | 4.50 | 14 | 1 | 9 | 82 | 92 | 6.57 | 10.9 | 1.46 |
| 30 | 12.62 | 14 | 2 | 8 | 43 | 53 | 3.79 | 18.9 | 0.30 |
| 32 | 9.06 | 14 | 7 | 17 | 45 | 69 | 4.93 | 34.8 | 0.54 |
| 33 | 7.07 | 14 | 7 | 9 | 22 | 38 | 2.71 | 42.1 | 0.38 |
| 34 | 10.94 | 7 | 0 | 6 | 41 | 47 | 6.71 | 12.8 | 0.61 |
| 35 | 9.82 | 4 | 0 | 1 | 10 | 11 | 2.75 | 9.1 | 0.28 |
| 36 | 5.37 | 14 | 3 | 12 | 32 | 47 | 3.36 | 31.9 | 0.63 |
| 37 | 7.82 | 14 | 2 | 26 | 96 | 124 | 8.86 | 22.6 | 1.13 |
| 39 | 9.54 | 14 | 7 | 44 | 94 | 145 | 10.36 | 35.2 | 1.09 |
| 40 | 5.01 | 14 | 1 | 7 | 19 | 27 | 1.93 | 29.6 | 0.38 |
| 42 | 3.80 | 14 | 4 | 11 | 40 | 55 | 3.93 | 27.3 | 1.03 |
| 43 | 3.09 | 14 | 2 | 5 | 17 | 24 | 1.71 | 29.2 | 0.55 |
| 44 | 8.19 | 14 | 3 | 12 | 42 | 57 | 4.07 | 26.3 | 0.50 |
| 45 | 14.80 | 14 | 2 | 22 | 56 | 80 | 5.71 | 30.0 | 0.39 |
| 46 | 25.28 | 14 | 5 | 42 | 94 | 141 | 10.07 | 33.3 | 0.40 |
| 53 | 15.48 | 14 | 10 | 37 | 89 | 136 | 9.71 | 34.6 | 0.63 |
| 54 | 6.90 | 14 | 1 | 13 | 15 | 29 | 2.07 | 48.3 | 0.30 |
| 58 | 1.70 | 14 | 0 | 0 | 2 | 2 | 0.14 | 0.0 | 0.08 |
| 59 | 6.68 | 13 | 1 | 6 | 13 | 20 | 1.54 | 35.0 | 0.23 |
| 60 | 12.38 | 8 | 0 | 4 | 6 | 10 | 1.25 | 40.0 | 0.10 |
| 61 | 4.30 | 13 | 3 | 5 | 30 | 38 | 2.92 | 21.1 | 0.68 |
| 64 | 3.72 | 13 | 1 | 27 | 101 | 129 | 9.92 | 21.7 | 2.67 |
| 65 | 11.90 | 13 | 7 | 41 | 128 | 176 | 13.54 | 27.3 | 1.14 |
| 66 | 8.44 | 4 | 0 | 1 | 10 | 11 | 2.75 | 9.1 | 0.33 |
| 67 | 3.12 | 14 | 2 | 20 | 39 | 61 | 4.36 | 36.1 | 1.40 |
| 68 | 7.26 | 14 | 5 | 8 | 38 | 51 | 3.64 | 25.5 | 0.50 |
| 69 | 10.44 | 13 | 11 | 34 | 88 | 133 | 10.23 | 33.8 | 0.98 |
| 71 | 24.82 | 4 | 0 | 4 | 30 | 34 | 8.50 | 11.8 | 0.34 |
| 73 | 11.82 | 7 | 2 | 7 | 25 | 34 | 4.86 | 26.5 | 0.41 |
| 74 | 11.61 | 14 | 4 | 20 | 37 | 61 | 4.36 | 39.3 | 0.38 |
| 76 | 16.30 | 14 | 5 | 16 | 38 | 59 | 4.21 | 35.6 | 0.26 |
| 77 | 17.92 | 11 | 8 | 31 | 54 | 93 | 8.45 | 41.9 | 0.47 |

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

| *Scheme* | *Scheme length (km)* | *Scheme years* | *Number of accidents* | | | | *Accident frequency (acc/year)* | *Accident severity (% fatal & serious)* | *Accident density (acc/year per km of scheme)* |
|---|---|---|---|---|---|---|---|---|---|
| | | | *Fatal* | *Serious* | *Slight* | *Total* | | | |
| 80 | 19.24 | 14 | 21 | 147 | 419 | 587 | 41.93 | 28.6 | 2.18 |
| 81 | 5.82 | 14 | 2 | 14 | 36 | 52 | 3.71 | 30.8 | 0.64 |
| 82 | 17.12 | 11 | 6 | 14 | 50 | 70 | 6.36 | 28.6 | 0.37 |
| 84 | 9.46 | 14 | 5 | 33 | 89 | 127 | 9.07 | 29.9 | 0.96 |
| 85 | 23.08 | 11 | 10 | 79 | 247 | 336 | 30.55 | 26.5 | 1.32 |
| 86 | 16.03 | 11 | 7 | 75 | 277 | 359 | 32.64 | 22.8 | 2.04 |
| 87 | 11.44 | 14 | 4 | 27 | 91 | 122 | 8.71 | 25.4 | 0.76 |
| 91 | 22.24 | 14 | 6 | 23 | 82 | 111 | 7.93 | 26.1 | 0.36 |
| 93 | 5.36 | 14 | 5 | 6 | 13 | 24 | 1.71 | 45.8 | 0.32 |
| 94 | 6.46 | 14 | 1 | 17 | 30 | 48 | 3.43 | 37.5 | 0.53 |
| 95 | 4.92 | 4 | 0 | 3 | 6 | 9 | 2.25 | 33.3 | 0.46 |
| 96 | 11.10 | 14 | 3 | 18 | 29 | 50 | 3.57 | 42.0 | 0.32 |
| 97 | 23.76 | 14 | 6 | 15 | 32 | 53 | 3.79 | 39.6 | 0.16 |
| 98 | 17.60 | 14 | 5 | 16 | 22 | 43 | 3.07 | 48.8 | 0.17 |
| 99 | 16.80 | 5 | 0 | 8 | 12 | 20 | 4.00 | 40.0 | 0.24 |
| 100 | 20.68 | 8 | 2 | 3 | 19 | 24 | 3.00 | 20.8 | 0.15 |
| 101 | 8.62 | 14 | 1 | 20 | 32 | 53 | 3.79 | 39.6 | 0.44 |
| 102 | 14.00 | 14 | 6 | 19 | 43 | 68 | 4.86 | 36.8 | 0.35 |
| 103 | 5.18 | 13 | 1 | 11 | 18 | 30 | 2.31 | 40.0 | 0.45 |
| 104 | 9.74 | 7 | 1 | 38 | 165 | 204 | 29.14 | 19.1 | 2.99 |
| 105 | 6.40 | 14 | 2 | 31 | 43 | 76 | 5.43 | 43.4 | 0.85 |
| 106 | 14.58 | 14 | 2 | 27 | 35 | 64 | 4.57 | 45.3 | 0.31 |
| 107 | 10.39 | 3 | 0 | 4 | 9 | 13 | 4.33 | 30.8 | 0.42 |
| 109 | 6.00 | 14 | 1 | 7 | 10 | 18 | 1.29 | 44.4 | 0.21 |
| 110 | 6.36 | 14 | 3 | 10 | 60 | 73 | 5.21 | 17.8 | 0.82 |
| 111 | 14.84 | 14 | 6 | 26 | 47 | 79 | 5.64 | 40.5 | 0.38 |
| 112 | 9.84 | 14 | 2 | 12 | 31 | 45 | 3.21 | 31.1 | 0.33 |
| 113 | 3.30 | 3 | 0 | 0 | 5 | 5 | 1.67 | 0.0 | 0.51 |
| 114 | 8.89 | 14 | 2 | 13 | 35 | 50 | 3.57 | 30.0 | 0.40 |
| 115 | 10.06 | 14 | 4 | 30 | 104 | 138 | 9.86 | 24.6 | 0.98 |
| 117 | 4.44 | 14 | 1 | 14 | 21 | 36 | 2.57 | 41.7 | 0.58 |
| 118 | 24.98 | 14 | 17 | 50 | 116 | 183 | 13.07 | 36.6 | 0.52 |
| 121 | 7.18 | 14 | 2 | 12 | 60 | 74 | 5.29 | 18.9 | 0.74 |
| 122 | 12.82 | 14 | 7 | 22 | 67 | 96 | 6.86 | 30.2 | 0.53 |
| 123 | 18.56 | 9 | 3 | 20 | 35 | 58 | 6.44 | 39.7 | 0.35 |
| 124 | 24.84 | 9 | 2 | 33 | 67 | 102 | 11.33 | 34.3 | 0.46 |
| 125 | 2.28 | 8 | 2 | 10 | 21 | 33 | 4.13 | 36.4 | 1.81 |
| 126 | 4.58 | 13 | 4 | 23 | 21 | 48 | 3.69 | 56.3 | 0.81 |
| 127 | 17.45 | 13 | 3 | 35 | 39 | 77 | 5.92 | 49.4 | 0.34 |
| 128 | 25.71 | 13 | 6 | 80 | 59 | 145 | 11.15 | 59.3 | 0.43 |
| 129 | 12.43 | 5 | 1 | 7 | 13 | 21 | 4.20 | 38.1 | 0.34 |
| 130 | 7.98 | 4 | 2 | 1 | 21 | 24 | 6.00 | 12.5 | 0.75 |
| 131 | 7.96 | 4 | 1 | 5 | 12 | 18 | 4.50 | 33.3 | 0.57 |
| 132 | 6.28 | 4 | 0 | 8 | 26 | 34 | 8.50 | 23.5 | 1.35 |
| 134 | 2.47 | 10 | 1 | 8 | 15 | 24 | 2.40 | 37.5 | 0.97 |
| 135 | 9.76 | 14 | 0 | 11 | 14 | 25 | 1.79 | 44.0 | 0.18 |
| 136 | 30.99 | 14 | 11 | 47 | 130 | 188 | 13.43 | 30.9 | 0.43 |
| 137 | 17.50 | 14 | 10 | 24 | 168 | 202 | 14.43 | 16.8 | 0.82 |
| 138 | 4.30 | 9 | 1 | 1 | 10 | 12 | 1.33 | 16.7 | 0.31 |
| 139 | 2.23 | 13 | 3 | 17 | 33 | 53 | 4.08 | 37.7 | 1.83 |
| 140 | 10.48 | 7 | 1 | 9 | 30 | 40 | 5.71 | 25.0 | 0.55 |
| 141 | 4.22 | 13 | 3 | 18 | 26 | 47 | 3.62 | 44.7 | 0.86 |
| 142 | 8.24 | 13 | 1 | 21 | 20 | 42 | 3.23 | 52.4 | 0.39 |
| 491 | 7.06 | 14 | 1 | 7 | 24 | 32 | 2.29 | 25.0 | 0.32 |
| 561 | 4.02 | 10 | 1 | 2 | 10 | 13 | 1.30 | 23.1 | 0.32 |
| 562 | 2.17 | 10 | 4 | 5 | 11 | 20 | 2.00 | 45.0 | 0.92 |
| Total | 1243.95 | 1298 | 421 | 2365 | 6310 | 9096 | 7.01 | 30.6 | 0.63 |

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

| *Summary variable* | | *Value* |
|---|---|---|
| Total length of schemes (1-direction) (km) | | 1244.0 |
| Total scheme-years | | 1298 |
| Total number of accidents | | 9097 |
| *of which:* | Fatal | 421 |
| | Serious | 2365 |
| | Slight | 6311 |
| Average accident frequency (acc/year/scheme) | | 7.0 |
| Average accident density (acc/year per km of scheme) | | 0.63 |
| Average accident severity (fatal and serious) | | 30.6% |

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

| *Location* | *Scheme years* | *Number of accidents* | | | | *Accident frequency (acc/year/ scheme)* | *Accident severity (% fatal & serious)* | *Accident density (acc/year per km of scheme)* |
|---|---|---|---|---|---|---|---|---|
| | | *Fatal* | *Serious* | *Slight* | *Total* | | | |
| Major junction | 1298 | 13 | 201 | 1064 | 1278 | 1.0 | 17 | 0.09 |
| Minor junction | 1298 | 92 | 539 | 1476 | 2107 | 1.6 | 30 | 0.15 |
| Link | 1298 | 316 | 1625 | 3771 | 5712 | 4.4 | 34 | 0.40 |
| Total | 1298 | 421 | 2365 | 6311 | 9097 | 7.0 | 31 | 0.63 |

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

| *Junction type* | *Number of junctions* | *Junction years* | *Number of accidents* | | | | *Accident frequency (acc/year/ junction)* | *Accident severity (% fatal & serious)* |
|---|---|---|---|---|---|---|---|---|
| | | | *Fatal* | *Serious* | *Slight* | *Total* | | |
| Roundabout, no signals | 53 | 525 | 9 | 167 | 883 | 1059 | 2.0 | 17 |
| Partially-signalled roundabout | 2 | 24 | 0 | 7 | 74 | 81 | 3.4 | 9 |
| Fully-signalled roundabout | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Junction with signals | 2 | 22 | 4 | 26 | 99 | 129 | 5.9 | 23 |
| Type not stated | | | 0 | 1 | 8 | 9 | | |
| All major junction accidents | 57 | 571 | 13 | 201 | 1064 | 1278 | 2.2 | 17 |
| Exit slip followed by entry slip | 155 | 1881 | 31 | 223 | 657 | 911 | 0.5 | 28 |
| T-junction with large island | 49 | 640 | 8 | 34 | 72 | 114 | 0.2 | 37 |
| T-junction with small or no island | 30 | 339 | 2 | 26 | 95 | 123 | 0.4 | 23 |
| Exit slip only | 61 | 713 | 9 | 57 | 150 | 216 | 0.3 | 31 |
| Entry slip only | 63 | 735 | 5 | 41 | 102 | 148 | 0.2 | 31 |
| Exit/entry on nearside with crossover | 21 | 249 | 6 | 13 | 42 | 61 | 0.2 | 31 |
| Crossover on offside to junction opposite | 21 | 253 | 1 | 12 | 26 | 39 | 0.2 | 33 |
| Staggered junction with crossover | 27 | 301 | 9 | 39 | 106 | 154 | 0.5 | 31 |
| Simple crossroads | 8 | 110 | 4 | 3 | 14 | 21 | 0.2 | 33 |
| Road leaves other road on exit slip | 2 | 23 | 1 | 1 | 0 | 2 | *(0.1)* | *(100)* |
| Road joins other road on entry slip | 2 | 23 | 1 | 4 | 3 | 8 | *(0.4)* | *(63)* |
| Other type of junction | 20 | 240 | 5 | 37 | 52 | 94 | 0.4 | 45 |
| Other type of junction with access | 20 | 233 | 7 | 31 | 82 | 120 | 0.5 | 32 |
| Type not stated | | | 3 | 18 | 75 | 96 | | |
| All minor junction accidents | 479 | 5740 | 92 | 539 | 1476 | 2107 | 0.4 | 30 |
| All junction accidents | 536 | 6311 | 105 | 740 | 2540 | 3385 | 0.5 | 25 |

*Brackets (…) indicate a figure based on too few accidents for ratios to be meaningful*

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

| Accident group | Number of accidents: Fatal | Serious | Slight | Total | Accident frequency (acc/year/scheme) | Accident severity (% fatal & serious) | % of all accidents |
|---|---|---|---|---|---|---|---|
| ***Junction accidents:*** | | | | | | | |
| A Pedestrian accident | 22 | 43 | 48 | 113 | 0.1 | 58 | 1 |
| B 1-vehicle | 15 | 192 | 481 | 688 | 0.5 | 30 | 8 |
| C 1-vehicle on internal link | 2 | 10 | 47 | 59 | 0.1 | 20 | 1 |
| D 2+ vehicle | 60 | 468 | 1886 | 2414 | 1.9 | 22 | 27 |
| E 2+ vehicle on internal link | 5 | 24 | 65 | 94 | 0.1 | 31 | 1 |
| All junction accidents | 104 | 737 | 2527 | 3368 | 2.6 | 25 | 37 |
| ***Link accidents:*** | | | | | | | |
| J Pedestrian accident | 74 | 101 | 88 | 263 | 0.2 | 67 | 3 |
| K Access accident | 7 | 26 | 57 | 90 | 0.1 | 37 | 1 |
| L 1-vehicle, left carriageway nearside on bend | 6 | 50 | 83 | 139 | 0.1 | 40 | 2 |
| M 1-vehicle, left carriageway nearside elsewhere | 29 | 252 | 628 | 909 | 0.7 | 31 | 10 |
| N 1-vehicle, left carriageway offside on bend | 2 | 28 | 42 | 72 | 0.1 | 42 | 1 |
| O 1-vehicle, left carriageway offside elsewhere | 12 | 151 | 332 | 495 | 0.4 | 33 | 5 |
| R 1-vehicle, other manoeuvre | 4 | 136 | 297 | 437 | 0.3 | 32 | 5 |
| S 2+ vehicle, one parked | 40 | 151 | 290 | 481 | 0.4 | 40 | 5 |
| T 2+ vehicle, one overtaking or changing lane | 28 | 253 | 662 | 943 | 0.7 | 30 | 10 |
| V 2+ vehicle, one stopped on carriageway | 10 | 85 | 453 | 548 | 0.4 | 17 | 6 |
| W 2+ vehicle, one turning or waiting to turn | 7 | 44 | 84 | 135 | 0.1 | 38 | 2 |
| X 2+ vehicle, one crossed central reservation | 47 | 92 | 110 | 249 | 0.2 | 56 | 3 |
| Y 2+ vehicle, going ahead on bend | 3 | 14 | 35 | 52 | 0.0 | 33 | 1 |
| Z 2+ vehicle, other manoeuvre | 47 | 239 | 605 | 891 | 0.7 | 32 | 10 |
| All link accidents | 316 | 1622 | 3766 | 5704 | 4.4 | 34 | 63 |
| Accidents not assigned to a group | 1 | 6 | 18 | 25 | | | |
| All accidents | 421 | 2365 | 6311 | 9097 | 7.0 | 31 | 100 |

accidents; it represents multi-vehicle accidents involving manoeuvres other than stopping, turning, overtaking or changing lane.

The groups with the highest severity were those involving pedestrians (67 per cent fatal or serious for pedestrian accidents on links, group J, and 58 per cent at junctions, group A), and multi-vehicle accidents where one vehicle crossed the central reservation (56 per cent fatal or serious, group X).

Table 11.6 presents the number of accidents classified as fatal, serious or slight, broken down by accident classes.

## 11.2 Breakdown of accidents

### *11.2.1 Accidents by number of casualties*

Table 11.7 shows the distribution of accidents by the number of casualties involved. A high proportion of accidents resulted in one casualty, with percentages of 66 per cent on links, 82 per cent at major junctions, and 68 per cent at minor junctions. These percentages of one-person injury accidents were all higher than the corresponding figures for single carriageways. The greatest number of casualties in a single accident was 20 (none fatal); this was a 2-vehicle accident involving a bus and a car. The next largest was 18, in an accident involving 17 vehicles.

The average number of casualties per accident was 1.58 for link accidents, 1.53 for accidents at minor junctions, and 1.26 at major junctions - all lower than on single carriageways. The number of fatal casualties per accident was higher in link accidents (0.07) and minor junction accidents (0.05) than at major junctions (0.01).

### *11.2.2 Accidents by number of vehicles and accidents involving pedestrians*

Table 11.8 analyses the accidents by the numbers of vehicles involved and the involvement of pedestrians. The majority of accidents (52 per cent) involved 2 vehicles; very few involved more than 3 vehicles. The highest number of vehicles involved in a single accident was 17.

The percentage of accidents involving pedestrians was comparatively small (4.3 per cent), with the highest being for link accidents (4.6 per cent). For comparison, on single carriageways 3.1 per cent (4.6 per cent of link accidents) involved pedestrians.

### *11.2.3 Vehicle involvements by type of vehicle and number of pedestrians involved*

Table 11.9 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; 70 per cent

**Table 11.6 Number, frequency, and severity of accidents by accident class**

| Accident class | Number of accidents | | | | Accident frequency (acc/year/ scheme) | Accident severity (% fatal & serious) | % of all accidents |
|---|---|---|---|---|---|---|---|
| | Fatal | Serious | Slight | Total | | | |
| ***Junction accidents:*** | | | | | | | |
| 1-vehicle | 18 | 203 | 531 | 752 | 0.6 | 29 | 8 |
| 2 vehicle | 46 | 422 | 1717 | 2185 | 1.7 | 21 | 24 |
| 3+ vehicle | 19 | 72 | 244 | 335 | 0.3 | 27 | 4 |
| All junction accidents | 83 | 697 | 2492 | 3272 | 2.5 | 24 | 36 |
| ***Link accidents:*** | | | | | | | |
| 1-vehicle | 53 | 618 | 1383 | 2054 | 1.6 | 33 | 23 |
| 2 vehicle | 107 | 540 | 1401 | 2048 | 1.6 | 32 | 23 |
| 3+ vehicle | 35 | 189 | 552 | 776 | 0.6 | 29 | 9 |
| All link accidents | 195 | 1347 | 3336 | 4878 | 3.8 | 32 | 54 |
| Not assigned to a class (pedestrian, access and stationary vehicle accidents) | 143 | 321 | 489 | 947 | 0.7 | 49 | 10 |
| All accidents | 421 | 2365 | 6311 | 9097 | 7.0 | 31 | 100 |

**Table 11.7 Accidents by number of casualties**

| Type | Number of accidents | Percent by number of casualties | | | | | | Average casualties/accident | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6+ | Fatal | Serious | Slight | Total |
| Major Junction | 1278 | 82 | 14 | 2 | 1 | 0 | 0 | 0.01 | 0.19 | 1.06 | 1.26 |
| Minor Junction | 2106 | 68 | 21 | 6 | 3 | 1 | 1 | 0.05 | 0.35 | 1.14 | 1.53 |
| Link | 5712 | 66 | 22 | 7 | 3 | 1 | 1 | 0.07 | 0.39 | 1.12 | 1.58 |
| Total | 9097 | 69 | 20 | 6 | 3 | 1 | 1 | 0.05 | 0.35 | 1.12 | 1.53 |

**Table 11.8 Accidents by number of vehicles and accidents involving pedestrians**

| Type | Number of accidents | Percent by number of vehicles | | | | | | Accidents involving pedestrians | |
|---|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6+ | Number of accidents | Percent of accidents |
| Major Junction | 1278 | 24 | 70 | 5 | 1 | 0 | 0 | 49 | 3.8 |
| Minor Junction | 2106 | 25 | 62 | 9 | 3 | 1 | 1 | 77 | 3.7 |
| Link | 5712 | 39 | 44 | 11 | 3 | 1 | 1 | 265 | 4.6 |
| Total | 9097 | 34 | 52 | 10 | 3 | 1 | 1 | 391 | 4.3 |

**Table 11.9 Vehicle involvements by type of vehicle, and pedestrian involvements**

| Type | Vehicle type | | | | | | Total vehicles involved | Pedestrians involved |
|---|---|---|---|---|---|---|---|---|
| | Pedal cycle | Motor cycle | Car/Taxi | Bus/Coach | LGV | HGV | | |
| Major Junction | 136 | 300 | 1587 | 25 | 117 | 162 | 2327 | 51 |
| Minor Junction | 182 | 308 | 2968 | 61 | 204 | 347 | 4070 | 78 |
| Link | 251 | 768 | 7455 | 140 | 689 | 1457 | 10760 | 288 |
| Total | 569 | 1376 | 12010 | 226 | 1010 | 1966 | 17157 | 417 |

of the 17157 vehicles involved in 9097 accidents were cars and taxis. More than half of the car involvements occurred on links, whereas more than half the pedal cycle accidents occurred at junctions. Buses and coaches had nearly twice as many involvements in link accidents as junction accidents, while HGVs had over three times as many.

## 11.3 Breakdown of accidents by time period

### *11.3.1 Variation of accident numbers by year*

The numbers of accidents by year (1979-1992) are shown in Table 11.10.

The growth in the number of accidents throughout the period did not give a true picture of the trend in accidents, because some schemes opened during the period. In order to estimate the actual time trend in accidents, a constant set of schemes was examined, namely those which opened before 1979 which therefore provided accident data throughout the period 1979-92. The growth rate in accident numbers on these schemes (calculated by fitting a log-linear regression line to the accident numbers) was 3.9 per cent per year for all accidents, and 3.0 per cent per year for link accidents alone, with 95 per cent confidence intervals of ± 2.0 per cent per year.

The growth rate in traffic over the same period was 4.5 per cent per year (Section 7.2). This implied a 1.5 per cent per year decrease in accident rate, with a 95 per cent confidence interval from 0.5 to 2.6 per cent per year decrease. Allowing for an additional error in the traffic growth estimate of ± 0.3 per cent per year (Section 7.2) increased the confidence range slightly to 0.4 to 2.6 per cent per year. It was therefore possible to say with some (but not high) statistical confidence that the underlying trend in accident rates was non-zero.

### *11.3.2 Variation of accident numbers by month of year*

Table 11.11 presents the distribution of accidents by month. July had the highest percentage of accidents (9.6 per cent), whereas February had the lowest (6.6 per cent). The range of variation was slightly less than that for single carriageways (6.3 to 10.4 per cent of accidents in the month).

**Table 11.11 Accidents by month of year**

| *Month* | *Number of accidents* | *Percent of accidents* | *Accident frequency (per year)* | *Accident ratio[1] Study schemes* | *Casualty ratio National Statistics[2]* |
|---|---|---|---|---|---|
| January | 719 | 7.9 | 6.7 | 0.95 | 0.87 |
| February | 600 | 6.6 | 5.6 | 0.79 | 0.81 |
| March | 648 | 7.1 | 6.0 | 0.86 | 0.93 |
| April | 647 | 7.1 | 6.0 | 0.86 | 0.90 |
| May | 737 | 8.1 | 6.8 | 0.98 | 1.01 |
| June | 756 | 8.3 | 7.0 | 1.00 | 1.01 |
| July | 873 | 9.6 | 8.1 | 1.16 | 1.07 |
| August | 851 | 9.4 | 7.9 | 1.13 | 1.07 |
| September | 813 | 9.0 | 7.5 | 1.08 | 1.02 |
| October | 773 | 8.5 | 7.2 | 1.02 | 1.14 |
| November | 851 | 9.4 | 7.9 | 1.13 | 1.14 |
| December | 799 | 8.8 | 7.4 | 1.06 | 1.04 |

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

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

The corresponding accident frequencies for dual carriageways were 8.1 and 5.6 accidents per scheme per year (this can be interpreted as the accident frequency for a year of Julys or Februarys respectively), compared to an overall average of 7.0.

Table 11.11 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, 1991), 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. The comparison shows that there were relatively more accidents in the summer, and fewer in the winter, on the rural trunk roads in the study than on roads in general.

**Table 11.10 Accidents by year**

| *Type* | *Year* 1979 | 1980 | 1981 | 1982 | 1983 | 1984 | 1985 | 1986 | 1987 | 1988 | 1989 | 1990 | 1991 | 1992 | *Total* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***All schemes:*** | | | | | | | | | | | | | | | |
| Major Junction | 23 | 38 | 49 | 78 | 69 | 52 | 82 | 126 | 128 | 108 | 134 | 112 | 148 | 131 | 1278 |
| Minor Junction | 75 | 82 | 82 | 134 | 104 | 144 | 133 | 169 | 156 | 214 | 171 | 214 | 201 | 228 | 2107 |
| Link | 269 | 285 | 269 | 391 | 333 | 351 | 401 | 430 | 418 | 464 | 563 | 545 | 485 | 508 | 5712 |
| Total | 367 | 405 | 400 | 603 | 506 | 547 | 616 | 725 | 702 | 786 | 868 | 871 | 834 | 867 | 9097 |
| ***Schemes open before 1979:*** | | | | | | | | | | | | | | | |
| Major Junction | 23 | 31 | 43 | 63 | 54 | 37 | 67 | 76 | 73 | 58 | 71 | 49 | 82 | 70 | 797 |
| Minor Junction | 75 | 81 | 72 | 101 | 77 | 109 | 97 | 123 | 105 | 148 | 102 | 138 | 146 | 142 | 1516 |
| Link | 269 | 266 | 238 | 327 | 266 | 281 | 315 | 316 | 299 | 328 | 378 | 379 | 346 | 375 | 4383 |
| Total | 367 | 378 | 353 | 491 | 397 | 427 | 479 | 515 | 477 | 534 | 551 | 566 | 574 | 587 | 6696 |

### *11.3.3 Variation of accident numbers by day of week*

Table 11.12 shows the distribution of accidents by day of week. Saturday had the highest percentage (17.2 per cent), whereas Wednesday had the lowest (12.7 per cent). The corresponding accident frequencies were 8.4 and 6.2 accidents per year, compared to an overall average of 7.0.

**Table 11.12 Accidents by day of week**

| *Day of week* | *Number of accidents* | *Percent of accidents* | *Accident frequency (per year)* | *Accident ratio[1] Study schemes* | *Driver involve -ment ratio National Statistics[2]* |
|---|---|---|---|---|---|
| Monday | 1255 | 13.8 | 6.8 | 0.97 | 0.97 |
| Tuesday | 1311 | 14.4 | 7.1 | 1.01 | 0.96 |
| Wednesday | 1177 | 12.9 | 6.4 | 0.90 | 0.98 |
| Thursday | 1154 | 12.7 | 6.2 | 0.89 | 1.05 |
| Friday | 1332 | 14.6 | 7.2 | 1.02 | 1.21 |
| Saturday | 1565 | 17.2 | 8.4 | 1.20 | 1.03 |
| Sunday | 1303 | 14.3 | 7.0 | 1.00 | 0.80 |

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

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

Like Table 11.11, Table 11.12 gives accident ratios, which can be compared with national statistics (Department of Transport, 1991) for driver involvements. The comparison shows that there were relatively more accidents at the weekend, and fewer towards the end of the week, on the rural trunk roads in the study than on roads in general.

# 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, hardstrips and safety fences 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 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.

In studies of accidents on single carriageways (including the TRL single-carriageway study), 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. It seems reasonable that the same variables should be investigated in this study of the variables which determine accident risk on dual-carriageway roads.

In the case of dual carriageways, the presence of kerb or hardstrip both on the nearside and on the median must be considered. The presence of a median safety fence was also included in this study as it was expected to be an important variable.

## 12.2 Methodology

The methods used for calculating hardstrip width and presence of kerbs and safety fences were discussed in Section 10. The way in which schemes were designated as either 2-lane (D2) or 3-lane (D3) dual carriageway was also discussed.

In this part of the analysis, accident data from only the last 5 available years (1988-1992) were used. For comparison with the single-carriageway study, accident rates over the 5-year period 1986-90 were also calculated. A 5-year period was used in order 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 (see Section 7.2) 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 only of those accidents classified as link accidents, and of the vehicle-km on the minor links only. 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 and on their internal links), and including the vehicle-km on the major links (including the internal links of minor junctions). This form of analysis gave rates which were comparable with those in earlier unpublished work for single-carriageway roads.

iii Scheme accident rates, that is, including all link, minor junction and major junction accidents, and including the vehicle-km on the whole scheme (including the internal links of junctions). This form of analysis was 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. It should be noted that major-link accident rates depend on the number of minor junctions occurring on each major link. Likewise, scheme rates depend 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 and all vehicle-km on internal links of junctions) | 0.104 accidents per million vehicle-km |
| Major-link rate: | (including accidents and vehicle-km on internal links at minor junctions, but not major junctions) | 0.124 accidents per million vehicle-km |
| Scheme rate: | (including accidents and vehicle-km on internal links at both major and minor junctions) | 0.143 accidents per million vehicle-km |

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 rates for Dual All-Purpose roads, which are 0.17 and 0.23 per MVkm, respectively. The rates derived here were substantially lower than the COBA-9 rates. However, many of the schemes in the sample were high standard rural dual carriageways with most or all junctions grade-separated, for which COBA-9 recommends a lower rate of 0.10 (for both link-only and combined link and junction rates), so the rates derived in this study lay within the COBA-9 range.

The spread of accident rates among the schemes studied was large. The link-only rates ranged from a minimum of 0.050 per MVkm to a maximum of 0.381 per MVkm (ignoring schemes with fewer than 20 accidents), compared with a mean of 0.104. The scheme rates ranged from a minimum of 0.051 per MVkm to a maximum of 0.602 (again ignoring schemes with fewer than 20 accidents), compared with a mean of 0.143.

In order to provide a comparison with rates from the single-carriageway study, the accident rates were recalculated using data for 1986-90. Table 12.2 shows a comparison between the three sets of mean rates: dual-carriageway accident rates using accident data for the latest 5 years (1988-92), dual-carriageway accident rates using data from 1986-90, and single-carriageway accident rates for 1986-90 from the earlier study.

The comparison between the two sets of rates for dual carriageways showed that link-only accident rates decreased from 0.111 to 0.104 over the two-year period, a decrease of 3 per cent per year. The decrease in major-link and scheme rates was similar. This trend was of a similar magnitude to that derived from time-trend analysis (see Section 13). A small part of the decrease could be ascribed to the lower contribution of schemes opened in or after 1986 to the former rate (see Section 12.4.1 below).

**Table 12.1 Averages and ranges for scheme accident rate (1988-1992)**

| | *Accident rates per 100MVehkm* | | | *Junction rates per 100MVeh* | |
|---|---|---|---|---|---|
| | *Link* | *Major link* | *Scheme* | *Major junction* | *Minor junction* |
| Average | 10.4 | 12.4 | 14.3 | 23.0 | 8.6 |
| Median | 9.8 | 12.1 | 13.2 | 0 | 6.2 |
| Lower quartile | 6.9 | 8.8 | 9.8 | 0 | 2.3 |
| Upper quartile | 13.7 | 16.8 | 19.4 | 8.3 | 11.1 |
| Minimum (excluding schemes with small numbers of accidents) | 5.0 | 5.1 | 5.1 | 8.8 | 3.0 |
| Maximum (excluding schemes with small numbers of accidents) | 38.1 | 42.4 | 60.2 | 49.2 | 42.0 |

*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 Major and minor junction vehicle throughput = the number of junctions times the total vehicle throughput, for calculation of junction accident rates*
*e Accident rates for links only, major links and schemes per 100 million vehicle-km.*
*f Accident rates for major and minor junctions per 100 million vehicles passing through the junctions.*
*g These results are based on the following data:*

| | | |
|---|---|---|
| *Number of schemes* | | *112* |
| *Number of Major junctions* | | *57* |
| *Number of Minor junctions* | | *431* |
| *Total length (km, 1-direction)* | | *1244.0* |
| *Total accident years* | | *544* |
| *Total traffic (MVehkm)* | | *29452.6* |
| *Number of accidents:* | *Link* | *2565* |
| | *Major junction* | *633* |
| | *Minor junction* | *1028* |

**Table 12.2 Comparison of accident rates, dual carriageways and single carriageways *(Duals accidents 1988-92 and 1986-90, singles accidents 1986-90)***

| | *Duals 1988-92* | *Duals 1986-90* | *Singles 1986-90* |
|---|---|---|---|
| Number of schemes | 112 | 112 | 108 |
| Accident years | 544 | 519 | 425 |
| ***Length (km, 1-direction):*** | | | |
| Total | 1244.0 | 1244.0 | 531.3 |
| Link | 1060.2 | 1060.2 | 531.3 |
| Major link | 1228.8 | 1228.8 | 531.3 |
| ***Vehicle-km (MVkm): (5-year period)*** | | | |
| Total | 29453 | 26055 | 7090 |
| Link | 24695 | 21792 | 7090 |
| Major link | 29053 | 25704 | 7090 |
| ***Junction throughput (MVkm): (5-year period)*** | | | |
| Major junction | 2749 | 2364 | 1105 |
| Minor junction | 12023 | 10711 | 3671 |
| ***Number of accidents: (5-year period)*** | | | |
| Total | 4226 | 3952 | 1640 |
| Link | 2565 | 2420 | 889 |
| Major junction | 633 | 608 | 240 |
| Minor junction | 1028 | 924 | 511 |
| Major link | 3593 | 3344 | 1400 |
| ***Accident rates (acc/MVkm):*** | | | |
| Link | 0.104 | 0.111 | 0.125 |
| Major link | 0.124 | 0.130 | 0.197 |
| Scheme | 0.143 | 0.152 | 0.231 |
| ***Accident rates (acc/Mveh throughput):*** | | | |
| Major junction | 0.230 | 0.257 | 0.217 |
| Minor junction | 0.086 | 0.086 | 0.139 |

The comparison between the duals and singles rates for 1986-90 showed that dual carriageways had a substantially lower accident rate than singles: about 11 per cent lower in the case of link accidents, and 34 per cent for major links and whole schemes, reflecting the higher incidence of minor junctions on single carriageways.

## 12.4 Results: Breakdown of accident rates

Table 12.3 presents a breakdown of link-only accident rates for 1988-92 by the main characteristics of scheme age, carriageway width, road quality and presence of kerbs, hardstrips and median safety fences. The significance level of the difference between the categories in each breakdown was tested with a $\chi^2$ test as indicated in the table. The presence of a kerb or hardstrip refers to the nearside of the carriageway, except where the table indicates that both sides were considered. The overall accident rate (row 1) was 0.104 per million vehicle-km.

### *12.4.1 Analysis by age*

Table 12.3 (row 2) shows that New (post-1986) schemes had an accident rate that was significantly lower at the 5 per cent level than older (Old or Mid) schemes. However, further down the table under 'By nearside kerb/age

**Table 12.3 Breakdown of accident rates by scheme characteristics (1988-1992) *Link-only accident rates except where specified***

| | *Accident rate (acc/MVkm)* | | | |
|---|---|---|---|---|
| | *Link* | *Major link* | *Scheme* | *Significance level* |
| ***1. Overall rate*** | 0.104 | | | |
| ***2. By age*** | | | | |
| Old | 0.106 | | | * |
| Mid | 0.103 | | | |
| New | 0.083 | | | |
| | *Newer schemes have lower accident rate* | | | |
| ***3. By median safety fence*** | | | | |
| No fence o/s | 0.098 | | | * |
| Fence o/s | 0.108 | | | |
| | *Schemes with safety fence have slightly higher rate* | | | |
| ***4. By median safety fence:*** | | | | |
| No fence o/s | 0.103 | | | NS |
| ***- D2 schemes only:*** | | | | |
| Fence o/s | 0.106 | | | |
| ***5. By carriageway width*** | | | | |
| 2-lane (D2) | 0.104 | | | NS |
| 3-lane (D3) | 0.102 | | | |
| ***6. By nearside hardstrip*** | | | | |
| No hardstrip n/s | 0.115 | | | *** |
| Hardstrip n/s | 0.099 | | | |
| | *Schemes with hardstrip have lower rate* | | | |
| ***7. By nearside kerb*** | | | | |
| No kerb n/s | 0.089 | | | *** |
| Kerb n/s | 0.117 | | | |
| | *Schemes with kerb have substantially higher rate* | | | |
| ***8. By road quality*** | | | | |
| Best | 0.100 | 0.110 | 0.110 | Link * |
| Good | 0.100 | 0.124 | 0.188 | |
| Lower | 0.110 | 0.149 | 0.199 | MjrL *** |
| Urban | 0.117 | 0.154 | 0.174 | Sch *** |
| | *Link: Urban schemes have higher rate*<br>*Major link and Scheme: Best<Good<Lower* | | | |
| ***9. By road quality - D2 schemes only:*** | | | | |
| Best | 0.104 | 0.115 | 0.115 | Link NS |
| Good | 0.100 | 0.124 | 0.188 | |
| Lower | 0.110 | 0.149 | 0.199 | Mjr L *** |
| Urban | 0.104 | 0.143 | 0.172 | Sch *** |
| | *Major link and Scheme: Best<Good<Lower* | | | |
| ***10. By nearside kerb/age combinations*** | | | | |
| No kerb, Old | 0.087 | | | *** by kerb |
| No kerb, Mid | 0.099 | | | NS by age |
| No kerb, New | 0.082 | | | |
| Kerb, Old | 0.119 | | | |
| Kerb, Mid | 0.107 | | | |
| Kerb, New | 0.092 | | | |
| | *No correlation between rate and age for schemes without kerb, or schemes with kerb* | | | |
| ***11. By nearside kerb/strip combinations*** | | | | |
| No kerb, No strip | 0.066 | | | *** by kerb |
| No kerb, strip | 0.089 | | | NS by strip |
| Kerb, No strip | 0.117 | | | |
| Kerb, strip | 0.118 | | | |
| | *No correlation between rate and presence of hardstrip for schemes without kerb, or schemes with kerb* | | | |

*Significance levels:* * *5 per cent level*
** *1 per cent level*
*** *0.1 per cent level*
*NS Not significant at the 5 per cent level*

combinations' (row 10) it is shown that when the presence of a nearside kerb was taken into account, there was no significant difference between the schemes according to their age. In other words, New schemes only appeared to have a lower rate because fewer of them had kerbs.

### *12.4.2 Analysis by presence of median safety fence*

The presence of an offside safety fence on the median (Table 12.3, row 3) was associated with an increase in the link-only accident rate from 0.098 to 0.108, a change which was significant but only at the 5 per cent level, and was in any case small. When only the 2-lane (D2) schemes were included (Table 12.3, row 4), there was no significant difference in accident rate between schemes with and without a safety fence.

It is rather surprising that schemes with a median safety fence should not have a lower accident rate. The result could be due to fences being installed predominantly in places where the risk was already higher than average, for example, to protect obstacles such as lamp-posts and bridge supports. It is reasonable to suppose that, in the absence of a safety fence, the number of accidents and their severity would be higher.

### *12.4.3 Analysis by carriageway width*

There was no significant difference between the overall average rates for 2-lane and 3-lane dual carriageways (Table 12.3, row 5).

### *12.4.4 Analysis by presence of hardstrip*

The presence of a 1-metre hardstrip on the nearside of the carriageway was associated with a reduction in the link-only accident rate from 0.115 to 0.099, a change which was significant at the 0.1 per cent level (Table 12.3, row 6). However, see 12.4.7 below.

### *12.4.5 Analysis by presence of kerb*

Schemes with a kerb on the nearside of the carriageway had a higher link-only accident rate of 0.117 as compared to a rate of 0.089 on those without, a difference that was significant at the 0.1 per cent level (Table 12.3, row 7). However, see 12.4.7 below.

### *12.4.6 Analysis by road quality*

The link-only accident rate for Urban Bypass quality roads (Table 12.3, row 8) was 0.117, which was significantly higher than for the other three qualities, which were not significantly different from each other. This difference was mainly due to the inclusion of a single 3-lane Urban Bypass scheme (scheme 80). When only the 2-lane schemes were included (Table 12.3, row 9), there were no significant differences between the road quality groups.

The analysis was repeated for major-link and scheme accident rates, as defined in section 12.3 (Table 12.3, row 8). The results showed that, on a major link basis, Best quality roads had a lower accident rate than Good quality roads, which in turn had a lower rate than Lower quality or Urban Bypass roads. The differences in each case were very highly significant (better than 0.1 per cent level). There was no significant difference between the rates for Urban Bypass and Lower quality roads. On a scheme basis, the results were similar, except that Urban Bypass roads had a significantly lower rate (at the 5 per cent level) than Lower quality roads. The same results occurred when only the 2-lane schemes were included (Table 12.3, row 9).

The conclusion was that, when junction accidents were taken into account, the overall accident rate was significantly lower on good modern roads because their junctions were fewer and of better design.

### *12.4.7 Analysis by width, kerb and hardstrip combinations*

In the sample of dual-carriageway schemes, there were correlations between the presence of kerbs and hardstrips, because both are features, and to a large extent alternative features, of the treatment of the edge of the carriageway. It was therefore necessary to consider combinations of the two. In Table 12.3 under 'By nearside kerb/strip combinations' (row 11) it is shown that when the presence of a nearside kerb was taken into account, there was no significant difference between the schemes according to the presence of a nearside hardstrip. However, this result conflicted with initial results from the regression analysis, which indicated that it was hardstrips rather than kerbs which were important (though there was a difference in that the regression analysis considered kerbs and hardstrips on both sides of the carriageway).

Because of the likelihood of correlations between carriageway width and the presence of kerbs and hardstrips, a more detailed analysis was carried out in which all combinations of carriageway width and the presence and absence of kerbs and hardstrips, nearside and offside, were included. This gave 32 possible combinations, of which 15 occurred in the sample of schemes. The accident rates for each combination are shown in Table 12.4. Combinations whose accident rates were not significantly different were progressively amalgamated, until a set of combinations which could not be further amalgamated was found. The results of this procedure were as follows.

**Table 12.4 Accident rates for combinations of kerb, hardstrip and carriageway widths (1988-1992)**
***Accidents per million vehicle-km***

| | *Hardstrips* | | | |
|---|---|---|---|---|
| | *Both* | *Left* | *Right* | *None* |
| ***Kerbs*** | | | | |
| Both | **D2** 0.162 | **D2** 0.137 | *No data* | **D2** 0.115<br>**D3** 0.148 |
| Left | **D2** 0.105 | **D2** *No data*<br>**D3** 0.127 | **D2** 0.079 | **D2** 0.120 |
| Right | **D2** 0.084 | **D2** 0.206 | *No data* | *No data* |
| None | **D2** 0.093<br>**D3** 0.061 | **D2** 0.157 | **D2** 0.088 | **D2** 0.057 |

*Where no accident rate is indicated for D3 schemes, there were no schemes in that category*

The rate for 3-lane (D3) schemes with hardstrips both sides but no kerbs was very highly significantly lower than the other two D3 rates. Where schemes with the same kerb and hardstrip configuration could be compared, both D3 rates were significantly different from the rates for D2 schemes (one larger, one smaller). This implied that D2 and D3 schemes could not be combined, nor could all D3 schemes be combined together. The result noted above in Section 12.4.3, that there was no significant difference between overall average rates for D2 and D3 schemes, arose because of averaging over rates which were significantly different.

For the D2 schemes, it was found that there were no differences between rows of the table, although there were significant differences between columns. The conclusion was that the presence of a kerb made no difference in any hardstrip configuration, but the presence of a hardstrip did make a difference. The significant effects of kerbs noted above were therefore due to correlations with the presence of hardstrips.

There was also no significant difference between the middle columns, so the categories of hardstrip left and right could be combined into one category for hardstrip one side only. In other words, if there were a hardstrip on only one side, it did not matter on which side (though most instances were on the nearside). The accident rate for this category was not significantly different from the category of no hardstrip.

The analysis produced two irreducible categories for each carriageway width with the following accident rates:

| | |
|---|---|
| Dual 2-lane (D2), hardstrip both sides: | 0.096 accidents per MVkm |
| Dual 3-lane (D3), hardstrip both sides: | 0.061 accidents per MVkm |
| Dual 2-lane (D2), hardstrip one or neither side: | 0.118 accidents per MVkm |
| Dual 3-lane (D3), hardstrip one or neither side: | 0.130 accidents per MVkm |

The latter two rates were not significantly different, and represented an average rate of 0.121 accidents per MVkm.

Therefore, schemes with hardstrips on both sides had a lower accident rate than schemes with a hardstrip on one side only or on neither side. Dual-3 schemes with hardstrips on both sides appeared to have a lower accident rate than similar dual-2 schemes; however, the very low accident rate of 0.061 per MVkm for dual-3 schemes was based on only 2 schemes with very high traffic levels, so was probably not typical of dual-3 schemes in general.

### 12.5 Conclusions from the tabular analysis

The main conclusion from this section is that the presence of a 1-metre hardstrip on both sides of the carriageway gave a significant reduction in link accident rates. The effect of a hardstrip on one side of the carriageway (normally the nearside) was similar to the effect of no hardstrips.

When the presence or absence of a kerb was allowed for, the age of the scheme or the presence of a safety fence had no effect on accident rate. Although the presence of a kerb on one or both sides appeared to increase the accident rates, this was due to a correlation with hardstrips, and when this was allowed for there was no evidence that kerbs as such had an effect on accident rates.

Accident rates on 3-lane dual carriageways appeared to be lower than on 2-lane dual carriageways, but in view of the small number of dual-3 schemes this conclusion is only tentative.

The conclusions on the effect of carriageway width, kerbs and hardstrips were broadly consistent with those from the regression analysis reported in Sections 15 and 16.

## 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, $\alpha$ and $\beta$ 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:

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

where $k_0$ is the base-year accident frequency parameter,

$Q_0$ is the base-year traffic flow,

$\theta$ is the underlying trend in accident risk,

$\gamma$ 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 value of the traffic growth parameter for all dual carriageways schemes together was estimated as 4.5 per cent per annum, and the average of the census-point-specific growth factors was 4.8 per cent per annum.

Equation (13.2) can be expressed as:

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

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 ($\theta$). In general, $\alpha$ 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:

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

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 ($\theta$) 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 $\theta$ 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:

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

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 Time trend analysis using global traffic growth

### *13.4.1 Results for dual carriageways*

Using an overall average value of 4.5 per cent for $\gamma$, the value of $\theta$ obtained from this analysis was -0.0215, implying a decrease in the accident risk factor *k* of 2.15 per cent per year. The 95 per cent confidence range for the decrease was from 1.2 to 3.0 per cent. This trend was non-zero with a high degree of significance. It was not significantly different from the 1.5 per cent decrease estimated in Section 11.3.1.

The value of $\alpha$ was 0.8876, and this was significantly different from 1 at the 0.1 per cent level. It was similar to that obtained from models with no time trend parameter.

### *13.4.2 Comparison with single-carriageway trend*

This decrease in accident risk of -2.15 per cent for dual carriageways could be compared to the time trend estimated by the same method in the single-carriageway study, which was found to be +1.7 per cent. However, an analysis of single-carriageway schemes opened in or before 1979 gave a similar value to the trend on dual carriageways, so there was an indication that the value obtained depended on the period over which the analysis was performed.

There are three possible explanations for this. One is that the time trend parameter itself varied with time, being around -2 per cent per annum in 1979 and increasing to around +2 per cent by 1992. Another way of expressing this is that the exponential function (equation 13.2) was not an adequate representation of the time trend effect.

Secondly, the growth rate for traffic flow might vary with time. Since a constant growth in traffic was assumed, any actual variation would transfer in the analysis to the trend in underlying accident rate. Traffic growth was in fact lower in the early 1980s than in the latter part of the decade, and there was no growth (or even a slight fall) between 1990 and 1992. However, the apparent variation in accident rate trend is too large to be accounted for fully by variation in traffic growth rate.

The third possible explanation is that older schemes had a different accident rate trend to newer ones, in other words that the accident rate trend depended on layout and design. This seems unlikely, but is possible. Such an effect would be allowed for automatically when design features were included in the model.

### *13.4.3 Changes in time trend with scheme age*

In order to investigate further, scheme age was used as a proxy for time period, and the time trend analysis was repeated with schemes broken down by opening year into the Old, Mid and New groups identified earlier. This was done for data from both the singles and duals studies, and gave the trends shown in Table 13.1.

The conclusion was that (apart from an anomalous result for single-carriageway schemes in the Mid age group) there were no significant differences between singles and duals in each age group. This indicates that the difference in trends between singles and duals could be ascribed to the larger proportion of older schemes in the duals study.

It was also found that the differences between the trends in the different age groups were not statistically significant, in spite of the apparently large differences in magnitude. The analysis in this report therefore proceeded on the basis of a common time trend across all age groups.

However, there was an indication, which was not statistically significant, that older schemes, or early years, had a negative trend and newer schemes, or later years, a positive trend. There is scope for further investigation to confirm this conclusion.

## 13.5 Time-trend analysis for 2- and 3-lane dual carriageways

In Section 12.4 it was found that the effect of features such as the presence of a hardstrip on accidents rates for 3-lane dual carriageways was very different from that for 2-lane dual carriageways, and this was confirmed in early regression models. It was therefore decided to model 2- and 3-lane roads (D2 and D3) separately. As a result it was necessary to re-run the time trend model separately for D2 and D3. The results were:

| | | |
|---|---|---|
| For D2 links: | $\alpha = 0.7713$ | (s.e. 0.0355) |
| | $\theta = -0.0183$ | (s.e. 0.0039) |
| For D3 links: | $\alpha = 1.451$ | (s.e. 0.173) |
| | $\theta = -0.0379$ | (s.e. 0.0082). |

Clearly, there were marked differences between the values obtained for D2 and D3 links, especially the $\alpha$ (traffic dependency) values but also the $\theta$ (time trend) values.

One possible explanation for the differences is that the D3 sample consisted only of a small number of links dominated by one particular scheme, scheme 80. This scheme had a particularly high traffic level, a large number of accidents, and a low traffic growth rate of about 2.5 per cent. Using a global rate of 4.5 per cent therefore overestimated traffic growth on this scheme, the effect of which was to over-estimate $\alpha$ (because there were more accidents than expected for the estimated traffic level) and to exaggerate the downward trend in underlying accident rate $\theta$ (because the actual number of accidents grew slower than expected for the estimated traffic level).

## 13.6 Time trend analysis using link-specific traffic growth

The effect described in the previous section could have been corrected by using a lower traffic growth factor for Scheme 80. However, there were no grounds for singling out one scheme for correction. It was therefore decided to calculate an individual traffic growth factor for every link, and repeat the time trend models. The individual traffic growth factors are referred to here as link-specific growth rates, but strictly speaking they were census-point-specific since the traffic flow on each link was taken as that at the nearest census point.

The results of fitting a time trend model using link-specific traffic growth factors were:

| | | |
|---|---|---|
| D2 links with link-specific | $\alpha = 0.8077$ | (s.e. 0.0357) |
| growth rate: | $\theta = -0.0225$ | (s.e. 0.0039) |
| D3 links with link-specific | $\alpha = 1.031$ | (s.e. 0.129) |
| growth rate: | $\theta = -0.0183$ | (s.e. 0.0083) |

A second calculation using individual traffic growth factors for each scheme, rather than for each link, gave very similar results.

**Table 13.1 Time trends by scheme age: single- and dual-carriageway comparison**

| *Carriageway type* | *Scheme age group* | *Trend (per cent per annum)* | *95 per cent confidence interval* |
|---|---|---|---|
| Dual carriageway | Old (1968-80) | -2.20 | -2.94 to -1.46 |
| | Mid (1981-85) | -1.33 | -3.95 to +1.29 |
| | New (1986-90) | +1.14 | -12.24 to +14.52 |
| | All | -2.15 | -2.85 to -1.45 |
| Single carriageway | Old (1968-80) | -2.27 | -4.89 to +0.35 |
| | Mid (1981-85) | +8.33 | +3.55 to +13.11 |
| | New (1986-90) | +3.51 | -10.97 to +17.99 |
| | All | +1.73 | -0.55 to +4.01 |

In these results, the values of $\alpha$ for D2 and D3 schemes were not significantly different (t-value 1.67). The value of $\alpha$ for D3 links was slightly greater than 1, but not significantly so (with a t-value of 0.24). The value of $\alpha$ for D2 links was highly significantly different from 1 (with a t-value of 5.39), and was near to the value found for all links and for single carriageways. These values were therefore more satisfactory than those obtained with a global traffic growth factor.

The values of $\theta$ for D2 and D3 links were not significantly different from each other (t-value 0.45). The value for D3 links was just significantly different from 0 (t-value 2.20), and the value for D2 schemes was highly significantly different from 0 (t-value 5.77). It was therefore not justified to assume a value of zero (no trend) as was done in the single-carriageway study.

### 13.7 Conclusions from time-trend analysis

The conclusion from this investigation of time-dependent accident risk was that, when different traffic growths on individual links were allowed for, 2- and 3-lane dual carriageways did not show significantly different trends. The underlying trend in accident risk on dual carriageways was a decrease of about 2 per cent per annum. Unlike in the singles study, the duals trend was highly significantly different from zero. There was therefore no justification for assuming no trend. However, the sampling errors and distributions of scheme ages were such that the time trends found in the singles and duals studies were not inconsistent.

From the analysis in Section 11.3.1 it was found that link-only accident numbers on a common subset of schemes (those opened before 1979) grew at a rate of 3.0 per cent per annum, compared with a 4.5 per cent growth in traffic. This implied a 1.5 per cent per annum decrease in link-only accident rate, with a 95 per cent confidence interval corresponding to a 0.4 to 2.6 per cent decrease, after allowing for an additional error of ± 0.3 per cent in the traffic growth estimate. This result was consistent with the results in this section.

It was decided to incorporate time trends into the regression modelling using the separate values for $\theta$ as derived above, that is -2.25 per cent for D2 links and -1.83 per cent for D3 links. However, as noted above, these values were not significantly different, so it would have been possible to use a common value of -2 per cent. D2 and D3 links would be modelled separately, and the modelling would use link-specific traffic growth factors.

## 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 Binch, 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, $\alpha$ and $\beta$ are parameters of the model.

If the value of $\beta$ 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 $\beta$ was around 0.7, and was statistically different from 1 at the 5 per cent level. $\beta$ 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. 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*

Sections 15 and 16 present 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. The tables give the value of *k* as $k_0$, the value appropriate to the base year (1990). For other years, this must be multiplied by $\exp(\theta t)$, where *t* is the year relative to 1990 (e.g. t=7 for 1997) and where θ is the time trend parameter, as explained in Section 13.

## 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*:

$$N_T = \int_T A\,dt = \int_T k_o.\exp(\theta t).[Q_o.\exp(\gamma t)]^{\alpha}.L_L^{\beta}.dt, \qquad (14.6)$$

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:

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

### *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:

$$\ln(N_T) = \ln(k_0) + \theta m + \alpha.\ln(Q_m) + \beta.\ln(L_L)$$
$$+ \ln(T) + (a_1A_1 + a_2A_2 + ...)$$
$$+ (d_1D_1 + d_2D_2 + ...). \qquad (14.9)$$

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 $[\theta m + ln(T)]$ 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 $d_i$.

### 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 $\chi^2$ with $(n-p-1)$ 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 $\chi^2$ 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 $\chi^2$. 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 $\chi^2$ 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 $3.8s$ and the true standard errors are estimated by multiplying the Poisson model standard errors by $s$. 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. As explained in Section 13.5 and 14.6, modelling was carried out separately for 2-lane and 3-lane links, using link-specific traffic growth factors and a time-trend parameter of -2.25 per cent for D2 links and -1.83 per cent for D3 links.

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 $\alpha$ (the traffic flow parameter) and $\beta$ (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 $\alpha$ (the traffic flow parameter) and $\beta$ (the link length parameter) were significantly different from 1. This was consistent with results from similar studies such as the single-carriageway study (Walmsley, Summersgill and Binch, 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, but it was decided to leave these variables free to take their own values in the modelling rather than force them to 1 in view of the general conclusion that accidents depend non-linearly with traffic and link length. In a few cases, very unusual results were obtained (such as very low or even negative values for $\alpha$ coupled with extreme values for other parameters). If it was found that a more robust model could be developed by setting $\alpha$ or $\beta$ to 1, this was done in those cases only.

#### *14.5.3 Correlation between variables*

In many of the models a number of variables were found to be significant but correlated with each other. For example, the variables relating to kerbs, hardstrips and continuous white lines all describe features of the edge treatment of a road, which tended to follow a small number of patterns according to the design specifications. It was therefore to some extent a matter of chance which edge treatment variable appeared as most significant. In some cases, these variables were combined into a multi-level factor which was then reduced by combining levels which were not statistically distinct; this process is described in the account for each model. In other cases, one variable was replaced in the model by another relating to that feature which perhaps had a slightly lower significance level but which was easier to interpret. In particular, the variable relating to the median verge width was defined in a way which made it easy to observe but difficult to interpret, so it was generally replaced in the models by a variable relating to the proportion of continuous obstruction on the right hand side of the carriageway.

#### *14.5.4 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 single-carriageway study (Walmsley, Summersgill and Binch, 1998), with some modifications, mainly additions, to allow for variables measured on both the nearside and the median side of the carriageway. The unit of analysis was the minor link, on one carriageway only.

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 5.5).

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.

D2 and D3 links were modelled separately. D3 modelling was confined to Level 1 and, at Level 2, to modelling by accident class, rather than by group. The modelling used link-specific traffic growth factors. Time trends were incorporated in the regression modelling using the separate values for $\theta$ of -2.25 per cent for D2 links and -1.83 per cent for D3 links, as derived in Section 13.

The models which were developed were:

***Level 1: Total accident-flow models:***
Analysis was carried out separately for 2-lane (D2) and 3-lane (D3) dual carriageways. Due to the relatively few D3 schemes and the low numbers of accidents, not all variables were tested.

1A. Total accidents as functions of traffic flow, number of years, time trend parameter and link length

1B. Total accidents as functions of traffic flow, number of years, time trend parameter and link length, plus the following major design variables:

Bendiness,

Hilliness,

Net gradient

Visibility (harmonic mean, maximum and minimum),

Opening date,

Presence of median safety fence (proportion, factor),

Link quality,

plus the following on both near- and offside:

Presence of verge (proportion, factor),

Mean verge width,

Presence of kerb (proportion, factor),

Presence of hardstrip (proportion, factor),

Presence of continuous edge marking (proportion, factor),

Presence of obstruction on verge or median (proportion, factor),

Number of accesses.

1C. Total accidents as functions of traffic flow, number of years, time trend parameter, link length and major design variables as in Level 1B, plus the following more detailed alignment variables:

Up-hilliness

Down-hilliness

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 (up or down) on link

Proportion of link with gradient greater than |4| per cent (desirable maximum for dual carriageways)

plus the following derived variables:

Speed estimate using COBA-9 formula (Department of Transport, 1981),

Speed estimate using COBA-10 formula (Lee and Brocklebank, 1993),

$V_{50,wet}$ (median speed of light vehicles on wet roads, as calculated using COBA-9 formula),

Design speed (calculated from $V_{50,wet}$).

***Level 2: Accident-flow models by accident group (carried out for all groups) and accident class:***

Analysis was carried out separately for 2-lane (D2) and 3-lane (D3) dual carriageways, but accidents on D3 links were relatively few in number and so were only broken down by class, not group.

2A. Accidents by group (D2 only) and class (D2 and D3) as functions of traffic flow, number of years and link length.

2B. Accidents by group (D2 only) and class (D2 and D3) as functions of traffic flow, number of years and link length, and major design variables as in Level 1B.

2C. Accidents by group (D2 only) and class (D2 and D3) as functions of traffic flow, number of years, time trend parameter, link length and major design variables as in Level 1B, plus more detailed alignment variables as in Level 1C.

In practice, Level 2B and 2C models were developed concurrently, and the results presented in Sections 15 and 16 are the combined results.

### 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 $A_{min}/A_{mean}$. The percentage change in number of accidents for a unit change in each explanatory variable is also shown in the tables.

## 15 Regression analysis results for dual-2 links

This section presents the detailed results from the regression analysis models for Dual-2 links, and the following section presents the results for Dual-3 links. 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 with the most important groups (generally, those with most accidents) first.

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 must be multiplied by exp(θt), where $t$ is the year relative to 1990 (e.g. t=7 for 1997), as explained in Section 13. The value of θ to be used for D2 links is -0.0225, representing a decrease in accident risk of 2.25 per cent per annum. In the logarithmic models below, the value of *ln(k)* should be taken as ($ln(k_0)$ - 0.0225$t$), 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.

Level 1A results for D2 links are presented in Table 15.1.

In most models the flow term parameter α and the length parameter β were significantly different from 1, implying, as expected, a non-linear relationship between accidents and flow, similar to that found in the single-carriageway study. The length parameter in particular, with a value of 0.695, indicated a strong non-linear relationship. The length parameter in the single-carriageway study (Walmsley, Summersgill and Binch, 1998) was 0.865.

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. This hypothesis could be explored by including a term such as ($L_L^{\beta}$ + a) in the model and testing whether β was significantly different from 1, but it was not possible to do this in the current study.

The best fit model at Level 1A was therefore:

$$\ln(A) = (-8.119 - 0.0225t) + 0.808.\ln(Q) + 0.695.\ln(L_L),$$

or, in its exponent form,

$$A = 2.98E\text{-}4.e^{-0.0225t}.Q^{0.808}.L_L^{0.695},$$

where E-4 denotes the multiplier $10^{-4}$. 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).

**Table 15.1 Level 1 models (D2 links): Total accidents as a function of traffic flow, link length and design variables**

| | | | | | *Multiplicative effect compared to mean at:* | | *% change in accidents for unit change in variable* | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| *Model* | *Model terms* | *Parameter value* | *s.e.* | *Deviance difference* | *Min* | *Max* | *Unit* | *%change* | *Residual deviance* | *Degrees of freedom* | *Scale factor* |
| ***Level 1A models: Total accidents as a function of traffic flow and link length (4688)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -0.341 | 0.0146 | | | | | | 3454 | 517 | 7.727 |
| Full | ln $k_0$ | -8.119 | 0.334 | | | | | | 1568 | 515 | 3.223 |
| | α | 0.808 | 0.0357 | 518 | | | | | | | |
| | β | 0.695 | 0.0180 | 1695 | | | | | | | |
| ***Level 1B models: Total accidents as a function of traffic flow, link length and major design variables (4688)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[d1.CROSS8 + d2.QUAL4 + a1.Bendi + a2.Hilli + a3.Obstrp_rhs + a4.Acc_rhs] | | | | | | | | | | | |
| Null | ln $k_0$ | -0.341 | 0.014 | | | | | | 3454 | 517 | 7.727 |
| Full | ln $k_0$ | -8.401 | 0.346 | | | | | | 1372 | 509 | 2.797 |
| | α | 0.804 | 0.0371 | 475 | | | | | | | |
| | β | 0.686 | 0.0222 | 1090 | | | | | | | |
| | CROSS8 | 0.272 | 0.0333 | 67 | | | Factor | +31% | | | |
| | QUAL4 | -0.290 | 0.0385 | 58 | | | Factor | -25% | | | |
| | Bendi | 0.00291 | 0.00068 | 18 | 0.90 | 1.59 | 10 deg/km | +2.9% | | | |
| | Hilli | 0.00881 | 0.00180 | 24 | 0.88 | 1.38 | 1 m/km | +0.9% | | | |
| | Obstrp_rhs | 1.440 | 0.305 | 19 | 0.99 | 1.95 | 10% | +15% | | | |
| | Acc_rhs | 0.0365 | 0.00946 | 14 | 0.96 | 1.44 | 1 access | +3.7% | | | |
| ***Level 1C models: Total accidents as a function of traffic flow, link length and extra design variables (4688)*** | | | | | | | | | | | |
| $A=k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[d1.CROSS8 + d2.QUAL4 + a1.Bendi + a2.Hilli + a3.Obstrp_rhs + a4.Acc_rhs] | | | | | | | | | | | |
| This model was the same as for Level 1B: no extra variables were significant | | | | | | | | | | | |

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

Level 1B models were derived by adding to the Level 1A model deduced above the major design variables listed in Section 14.6, using a stepwise fitting procedure. The results for D2 links are included in Table 15.1. Each variable was tested for significant effect when it was tried in the model alone. The most significant variable was then incorporated in the model, and the remaining variables again tested for significance. This was repeated until no more variables were significant. At each stage all previous variables in the model were tested to ensure that they remained significant in the developing model.

The most significant variables at Level 1B were found to be those relating to the presence of kerbs and hardstrips, for either side of the carriageway. As was found in the analysis of accident rates in Section 12.4, these variables were highly correlated, so it was necessary to consider them together in various combinations. This was done by including a multi-level factor in the modelling, and then combining those levels which were not statistically distinct. This process resulted in a 2-level factor CROSS8 defined as follows:

*CROSS8 level 1:* Hardstrips on both sides regardless of kerbing;
Hardstrip on one side only with no kerbing;
Hardstrip on one side only with kerbing on the same side.

*CROSS8 level 2:* No kerbs or hardstrips on either side;
Kerbing only;
Hardstrip on one side only with kerbing on opposite side of the road.

The two level factor CROSS8 represents the various combinations of hardstrip and kerb on the right and left hand side of the carriageway. Level 1 represents links where any kerbs present are set back behind hardstrips. In contrast, level 2 represents links where the carriageway edge is level or with kerbs (not set back behind a hardstrip). The latter links generated 31.3 per cent more accidents.

This result was similar to, but not entirely consistent with, that found in the tabular analysis (Section 12.4.7). The difference occurred because regression analysis takes account of non-linearities in the dependence of accidents on traffic flow and geometric factors. The difference was in any case not great because the links classified as CROSS8 level 1 were predominantly in the group with hardstrips both sides.

The next most significant variable was a two level factor QUAL4 representing the quality of the links, level 1 of the factor being Best or Good quality, and level 2 being Lower or Urban quality. As defined in Section 10.4.2, Best and Good quality links are good high speed modern roads of near motorway standard, where any minor junctions are mostly grade separated or have long splayed side roads. Good quality roads also have some large roundabouts with long curved approaches. Lower quality are older style roads with minor junctions often T-junctions or crossroads with only short deceleration lanes, and with major junctions which are smaller roundabouts with straight approaches and shorter distances between them. Urban quality roads are similar to Good quality, but with frequent minor junctions with deceleration lanes and often frequent roundabouts.

The model indicated that Urban or Lower quality roads had 25.1 per cent fewer link accidents than Best or Good quality roads. This was a surprising result, as the tabulation analysis (section 12.4.6) implied that there were no significant differences between the quality groups. It was, however, a robust result as it also appeared in many Level 2 models. It may well be related to the higher speeds attained on better quality roads. Better quality roads would probably carry more traffic which, through the non-linearity effect, would lead to a lower accident rate, which would tend to conceal a difference in the tabulation analysis. The tabulation analysis, however, also showed that major-link and scheme accident rates were lower on better-quality roads (see section 12.4.6). This led to the conclusion that roads of better design (generally, more modern roads) had fewer accidents because their junctions were fewer and of better design.

There was some evidence of a correlation between the factor QUAL4 representing road quality and the factor CROSS8 representing kerb and hardstrip combinations; this was to be expected as better quality roads were more likely to have hardstrips. However, for ease of interpretation, these factors were retained separately in the models.

The remaining variables which were found to be significant were bendiness (Bendi), hilliness (Hilli) (there was some correlation between bendiness and hilliness), the number of accesses on the right hand side (Acc_rhs), and the proportion of link with a continuous obstruction on the right hand (median) side (Obstrp_rhs). A continuous obstruction is an obstacle which would be hit if a vehicle left the carriageway at any point, including walls, woodland, slopes and safety fences, but excluding isolated items of street furniture. There was little evidence of a correlation between Obstrp_rhs and CROSS8, though both refer to the offside edge of the carriageway.

The value of the bendiness parameter was 0.00291. An increase in bendiness of 10 deg/km in the model increased accidents by 2.9 per cent. Bendiness ranged between 0.0 and 195.1 deg/km, with mean 36.1, for the 518 links.

The value of the hilliness parameter was 0.00881. An increase in hilliness of 1 metre/km in the model increased accidents by 0.9 per cent. Hilliness ranged between 0.0 and 50.7, with mean 14.2, for the 518 links.

The value of the parameter for the proportion of link with a continuous obstruction on the right hand side of the carriageway was 1.440. A link with a continuous obstruction on the right hand side for a tenth of its length generated 15.5 per cent more accidents in the model than a link with no obstructions. The proportion ranged from 0.0 to 0.47 with mean 0.005 for the 518 links.

The value of the access parameter was 0.0365. An additional access on the right hand side of the road increased accidents in the model by 3.7 per cent. The number of accesses on the right hand side ranged between

0 and 11, with mean 1, for the 518 links. Strictly speaking, a more useful parameter would be the number of accesses per km of link; however, when this was tested in preliminary models it was found not to give a significantly better fit than the number alone.

The final Level 1B model for D2 links was therefore:

$$\ln(A) = (-8.401 - 0.0225t) + 0.804.\ln(Q) + 0.686.\ln(L_L) + d1.CROSS8 + d2.QUAL4 + 0.00291.Bendi + 0.00881.Hilli + 1.440.Obstrp\_rhs + 0.0365.Acc\_rhs,$$

or, in its exponent form,

$$A = 2.25E\text{-}4.e^{-0.0225t}.Q^{0.804}.L_L^{0.686}.\exp(d1.CROSS8 + d2.QUAL4 + 0.00291.Bendi + 0.00881.Hilli + 1.440.Obstrp\_rhs + 0.0365.Acc\_rhs),$$

| | | |
|---|---|---|
| where | d1 = 0 | for level 1 of CROSS8 (Any kerbs set-back behind hardstrips) |
| | d1 = 0.272 | for level 2 of CROSS8 (Kerbs at carriageway edge) |
| | d2 = 0 | for level 1 of QUAL4 (Best or Good quality) |
| | d2 = -0.290 | for level 2 of QUAL4 (Lower or Urban quality) |

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).

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

In Level 1C, fifteen extra variables, comprising more detailed alignment measures and estimates of vehicle speed, as listed in Section 14.6, were added to the final Level 1B model. The alignment variables were tested in a step-wise regression procedure as for other 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.

None of the extra alignment variables was found to be significant at the 5 per cent level. Up-hilliness and Down-hilliness were closely correlated with Hilliness, which was already included in the model, so they added no extra explanatory power. With Hilliness removed, Down-hilliness became significant at the 5 per cent level, but did not improve the fit of the model compared to Hilliness, so the latter was retained.

None of the speed variables was found to be significant at the 5 per cent level. This was because the speed formulae for dual carriageways were essentially linear combinations of Bendiness and Hilliness, which were already included in the model, so they added no extra explanatory power. With Bendiness removed, $V_{50,wet}$ and Design speed became significant at the 5 per cent level, but did not improve the fit of the model compared to Bendiness, so the latter was retained.

It was noted in Section 15.2 that the reason why Good and Best quality links had a higher accident frequency could be that speeds were higher on such links. This raises the question as to why, when speed as such was included in the model, it was not significant. There are two possible explanations for this. One is that the formulae used for speed did not adequately represent actual vehicle speeds, and that the quality variable was a better estimator of speed. This could be true since the formulae effectively only take account of road alignment, whereas the quality parameter mainly depends on the number and types of junctions. The other possibility is that speed was correlated with quality and, since quality was already included in the models as a Level 1B parameter, adding speed in Level 1C did not significantly improve the fit. This could be tested, as with bendiness, by testing a model which included speed but not quality, but this was not done.

The conclusion was that the best fit Level 1C model was the same as that for Level 1B.

## 15.4 Level 2 models for multi-vehicle accident groups: Accidents by 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 using traffic flow and link length as explanatory variables (with accident years included as an offset throughout), plus all the design features listed in Section 14.6 (that is, the same variables as at Levels 1B and 1C).

The Level 2 model results for D2 links and multi-vehicle accidents (accident groups S to Z) are shown in Table 15.2.

### *15.4.1 Group T: Multi-vehicle accidents involving one vehicle overtaking or changing lane*

There were 741 accidents in the group. The most significant variables (at the 1 per cent level) were those relating to verge width on the right hand side and proportion of link with a continuous obstruction on the right hand side, Obstrp_rhs. As explained previously, there were problems with the interpretation of median verge width, so Obstrp_rhs was accepted into the model.

The next most significant variables (at the 1 per cent level) were kerb and hardstrip variables. Since kerbs and hardstrips were correlated, a factor representing the various combinations of edge treatment was developed, and was reduced (by eliminating levels which were not statistically distinct) to the factor used in Level 1B, CROSS8. The Quality factor was significant (at the 5 per cent level) and was reduced to the 2-level factor QUAL4 used in Level 1B.

**Table 15.2 Level 2 models (D2 links) for multi-vehicle accident groups: Accidents by group as a function of traffic flow, link length and major design variables**

| Model | Model terms | Parameter value | s.e. | Deviance difference | Multiplicative effect compared to mean at: Min | Max | % change in accidents for unit change in variable: Unit | %change | Residual deviance | Degrees of freedom | Scale factor |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Group T: Multi-vehicle Accidents involving one vehicle overtaking or changing lane (741)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -2.186 | 0.0367 | | | | | | 990.8 | 517 | 2.009 |
| 2A | ln $k_0$ | -10.74 | 0.838 | | | | | | 719.5 | 515 | 1.366 |
| | $\alpha$ | 0.896 | 0.0892 | 101.6 | | | | | | | |
| | $\beta$ | 0.629 | 0.0445 | 224.1 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a3.Obstrp_rhs + d1.CROSS8 + d2.QUAL4] | | | | | | | | | | | |
| Full | ln $k_0$ | -10.55 | 0.861 | | | | | | 689.3 | 512 | 1.310 |
| | $\alpha$ | 0.866 | 0.0929 | 87.7 | | | | | | | |
| | $\beta$ | 0.627 | 0.0491 | 189.1 | | | | | | | |
| | Obstrp_rhs | 2.809 | 0.604 | 15.9 | 0.99 | 3.69 | 10% | +32% | | | |
| | CROSS8 | 0.308 | 0.0820 | 14.0 | | | Factor | +36% | | | |
| | QUAL4 | -0.233 | 0.0921 | 6.5 | | | Factor | -21% | | | |
| ***Group V: Multi-vehicle Accidents involving one vehicle stopping, starting or held up (419)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -2.756 | 0.0487 | | | | | | 917.0 | 517 | 2.622 |
| 2A | ln $k_0$ | -15.40 | 1.11 | | | | | | 721.8 | 515 | 2.303 |
| | $\alpha$ | 1.337 | 0.118 | 129.2 | | | | | | | |
| | $\beta$ | 0.590 | 0.0586 | 112.3 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a3.Obstrp_rhs + a5.Kerbp_rhs + d2.QUAL4] | | | | | | | | | | | |
| Full | ln $k_0$ | -15.23 | 1.16 | | | | | | 692.3 | 512 | 1.877 |
| | $\alpha$ | 1.305 | 0.124 | 110 | | | | | | | |
| | $\beta$ | 0.587 | 0.0646 | 93.9 | | | | | | | |
| | Obstrp_rhs | 3.184 | 0.735 | 12.5 | 0.98 | 4.39 | 10% | +37% | | | |
| | Kerbp_rhs | 0.462 | 0.123 | 14.1 | 0.83 | 1.31 | 10% | +5% | | | |
| | QUAL4 | -0.361 | 0.121 | 9.1 | | | Factor | -30% | | | |
| ***Group W: Multi-vehicle Accidents involving one vehicle turning or waiting to turn (129)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -3.934 | 0.0872 | | | | | | 478.2 | 517 | 1.665 |
| 2A | ln $k_0$ | -7.03 | 1.92 | | | | | | 458.9 | 515 | 1.660 |
| | $\alpha$ | 0.319 | 0.207 | 2.3 | | | | | | | |
| | $\beta$ | 0.419 | 0.0989 | 18.9 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a4.Acc_rhs] | | | | | | | | | | | |
| Full | ln $k_0$ | -13.52 | 0.113 | | | | | | 460.5 | 515 | 2.126 |
| | $\alpha$ | 1 | 0 | | | | | | | | |
| | $\beta$ | 0.349 | 0.108 | 10.7 | | | | | | | |
| | Acc_rhs | 0.150 | 0.0489 | 8.9 | 0.85 | 4.93 | 1 access | +17% | | | |
| ***Group X: Multi-vehicle Accidents involving one vehicle crossing the central reservation (238)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -3.321 | 0.0646 | | | | | | 616.8 | 517 | 1.567 |
| 2A | ln $k_0$ | -8.48 | 1.48 | | | | | | 496.8 | 515 | 1.312 |
| | $\alpha$ | 0.512 | 0.158 | 10.4 | | | | | | | |
| | $\beta$ | 0.835 | 0.0818 | 119.4 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[d3.CROSSQU7 + a2.Hilli + d4.CRASHF] | | | | | | | | | | | |
| Full | ln $k_0$ | -9.94 | 1.51 | | | | | | 459.8 | 512 | 1.225 |
| | $\alpha$ | 0.630 | 0.159 | 15.4 | | | | | | | |
| | $\beta$ | 0.914 | 0.0862 | 130.5 | | | | | | | |
| | CROSSQU2 | 0.651 | 0.135 | 21.4 | | | Factor | +92% | | | |
| | Hilli | 0.0202 | 0.0077 | 6.5 | 0.75 | 2.09 | 1 m/km | +2% | | | |
| | CRASHF | -0.400 | 0.136 | 8.8 | | | Factor | -33% | | | |

**Table 15.2 Level 2 models (D2 links) for multi-vehicle accident groups: Accidents by group as a function of traffic flow, link length and major design variables (continued)**

| *Model* | *Model terms* | *Parameter value* | *s.e.* | *Deviance difference* | *Multiplicative effect compared to mean at:* Min | Max | *% change in accidents for unit change in variable* Unit | %change | *Residual deviance* | *Degrees of freedom* | *Scale factor* |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Group Y: Multi-vehicle Accidents, going ahead on a bend (47)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -4.943 | 0.145 | | | | | | 242.9 | 517 | 1.328 |
| 2A | ln $k_0$ | -8.89 | 3.19 | | | | | | 239.0 | 515 | 1.289 |
| | $\alpha$ | 0.419 | 0.343 | 1.5 | | | | | | | |
| | $\beta$ | 0.275 | 0.159 | 3.1 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a6.Propc_d + d2.QUAL4] | | | | | | | | | | | |
| Full | ln $k_0$ | -14.99 | 0.327 | | | | | | 220.3 | 514 | 1.009 |
| | $\alpha$ | 1 | 0 | | | | | | | | |
| | $\beta$ | 0.434 | 0.172 | 6.8 | | | | | | | |
| | Propc_d | 2.315 | 0.538 | 18.5 | 0.48 | 4.86 | 10% | +26% | | | |
| | QUAL4 | -0.762 | 0.353 | 4.9 | | | Factor | -53% | | | |
| **Group Z: Multi-vehicle Accidents, other manoeuvre (658)** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -2.304 | 0.0389 | | | | | | 1074.0 | 517 | 2.432 |
| 2A | ln $k_0$ | -13.35 | 0.902 | | | | | | 727.3 | 515 | 1.536 |
| | $\alpha$ | 1.154 | 0.0955 | 147.1 | | | | | | | |
| | $\beta$ | 0.758 | 0.0488 | 276 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[d5.QUAL6 + d6.KERBF_lhs + a7.Crash_p] | | | | | | | | | | | |
| Full | ln $k_0$ | -13.10 | 0.927 | | | | | | 696.6 | 512 | 1.499 |
| | $\alpha$ | 1.114 | 0.0994 | 125.0 | | | | | | | |
| | $\beta$ | 0.745 | 0.0515 | 247.7 | | | | | | | |
| | QUAL6 | -0.325 | 0.0831 | 15.3 | | | Factor | -28% | | | |
| | KERBF_lhs | 0.305 | 0.0850 | 13.1 | | | Factor | +36% | | | |
| | Crash_p | 0.208 | 0.0857 | 5.9 | 0.91 | 1.16 | 10% | +2% | | | |
| **Group S: Multi-vehicle Accidents, one vehicle parked (406)** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -2.787 | 0.0495 | | | | | | 863.9 | 517 | 2.101 |
| 2A | ln $k_0$ | -11.92 | 1.16 | | | | | | 606.8 | 515 | 1.480 |
| | $\alpha$ | 0.935 | 0.123 | 58.3 | | | | | | | |
| | $\beta$ | 0.924 | 0.0644 | 240.4 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a4.Acc_rhs + d2.QUAL4 + a8.Dnhilli + a9.Propc_c] | | | | | | | | | | | |
| Full | ln $k_0$ | -12.99 | 1.18 | | | | | | 577.4 | 511 | 1.381 |
| | $\alpha$ | 1.022 | 0.125 | 68.4 | | | | | | | |
| | $\beta$ | 0.752 | 0.0736 | 118.3 | | | | | | | |
| | Acc_rhs | 0.094 | 0.0282 | 10.0 | 0.91 | 2.55 | 1 access | +9.8% | | | |
| | QUAL4 | -0.370 | 0.135 | 7.7 | | | Factor | -31% | | | |
| | Dnhilli | 0.0167 | 0.00633 | 6.5 | 0.89 | 2.01 | Factor | +1.7% | | | |
| | Propc_c | 0.613 | 0.262 | 5.4 | 0.82 | 1.52 | Factor | +6.3% | | | |

The best fit model was:

$$\ln(A) = (-10.551 - 0.0225t) + 0.866.\ln(Q) + 0.627.\ln(L_L) + 2.809.Obstrp\_rhs + d1.CROSS8 + d2.QUAL4,$$

or, in its exponent form,

$$A = 2.62E\text{-}5.e^{0.0225t}.Q^{0.866}.L_L^{0.627}.\exp(2.809.Obstrp\_rhs + d1.CROSS8 + d2.QUAL4),$$

where d1 = 0 for level 1 of CROSS8 (Any kerbs set-back behind hardstrips)

d1 = 0.308 for level 2 of CROSS8 (Kerbs at carriageway edge)

and d2 = 0 for level 1 of QUAL4 (Best or Good quality)

d2 = -0.233 for level 2 of QUAL4 (Lower or Urban quality).

The flow parameter α was not significantly different from 1 but the length parameter β was significantly different (at the 0.1 per cent level).

The model suggested that overtaking and changing lane accidents were affected by the quality of road, the edge treatment and the proportion of link with an obstruction on the right hand side of the road.

The parameter of Obstrp_rhs was 2.809. A link with a continuous obstruction along a tenth of its length on the right hand side would have 32.4 per cent more accidents than a link with no obstructions. This figure is relatively large but is unrepresentative as the mean value of Obstrp_rhs was only 0.5 per cent; a link with this mean proportion of obstruction would have 1.4 per cent more accidents than one with none.

The CROSS8 factor suggested that links where the carriageway edge was level or with kerbs which were not set back behind a hardstrip would have 36.1 per cent more accidents than links where any kerbs present were set back behind hardstrips.

The QUAL4 factor suggested that Lower or Urban quality roads had 20.8 per cent fewer accident than Best or Good quality roads. This may well be related to the overtaking opportunities on the different quality roads. The higher quality roads have longer stretches between major and minor junctions and hence more overtaking opportunity. Speeds also tend to be higher on better quality roads. However, as with Level 1C models, the speed parameter itself was not found to be significant.

### *15.4.2 Group V: Multi-vehicle Accidents involving one vehicle stopping, starting or held up*

There were 419 accidents in the group. The most significant variable (at the 1 per cent level) was the proportion of link with a continuous obstruction on the right hand side, Obstrp_rhs. The next most significant (at the 1 per cent level) were the proportion of kerb and of continuous white line on the right hand side; neither was significant when the other was included, so the proportion of kerb (Kerbp_rhs) was used in the model. The Quality factor was significant (at the 5 per cent level) and was reduced to the 2-level factor QUAL4 used in Level 1B.

The best fit model was:

$$\ln(A) = (-15.23 - 0.0225t) + 1.305.\ln(Q) + 0.587.\ln(L_L) + d2.QUAL4 + 3.184.Obstrp\_rhs + 0.462.Kerbp\_rhs,$$

or, in its exponent form,

$$A = 2.43E\text{-}7.e^{-0.0225t}.Q^{1.305}.L_L^{0.587}.\exp(d2.QUAL4 + 3.184.Obstrp\_rhs + 0.462.Kerbp\_rhs),$$

where d2 = 0 for level 1 of QUAL4 (Best or Good quality)

d2 = -0.361 for level 2 of QUAL4 (Lower or Urban quality).

The flow parameter α was not significantly different from 1 but the length parameter β was significantly different (at the 0.1 per cent level).

The model suggested that the Lower or Urban quality links had 30.3 per cent fewer accidents than the Best or Good quality links. Again this may well be due to the speeds attained on the higher quality roads where a greater stopping distance would be required.

The obstruction parameter 3.184 suggested that a link with a continuous obstruction on the right hand side along a tenth of its length would have 37.5 per cent more accidents than a link with no obstructions on the right hand side.

The kerb parameter suggested that kerbs on the right hand side of the carriageway had an adverse effect on accidents. A link with kerb along a tenth of it length would have 4.7 per cent more accidents than a link with no kerb.

The presence of the kerb and obstruction parameters in the model might indicate that there was a particular problem when a vehicle was held up in the right hand lane and a kerb or obstruction was also present.

### *15.4.3 Group W: Multi-vehicle accidents involving one vehicle turning or waiting to turn*

There were 129 accidents in the group. This model did not give robust results, probably due to the relatively low number of accidents. Initially, the traffic flow parameter α was not significantly different from 0, which implied that accidents in this group were independent of traffic levels, which was not reasonable. The flow parameter was therefore constrained to $\alpha = 1$. The only significant variable (at the 5 per cent level) was the number of accesses on the right hand side; in the unconstrained model, accesses on the left hand side were also significant (at the 5 per cent level). In both cases, there was a correlation with link length. The best fit model was:

$$\ln(A) = (-13.52 - 0.0225t) + \ln(Q) + 0.349.\ln(L_L) + 0.150.Acc\_rhs,$$

or, in its exponent form,

$$A = 1.34E\text{-}6.e^{-0.0225t}.Q.L_L^{0.349}.\exp(0.150.Acc\_rhs).$$

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

The model suggested that accesses on the right hand side did have an effect on turning or waiting to turn accidents, as expected. An additional access on the link would increase accidents by 17.3 per cent.

### 15.4.4 *Group X: Multi-vehicle accidents involving one vehicle crossing the central reservation*

There were 238 accidents in the group. Quality and presence of hardstrip were the most significant variables (at the 1 per cent level), but these were correlated, so a factor combining the two was developed and was reduced to a 2-level factor CROSSQU7 as follows (unlike earlier models, the interactions between hardstrips, as in CROSS8, and quality, as in QUAL4, were significant):

| | |
|---|---|
| CROSSQU7 level 1 | Lower quality links |
| | Urban quality links |
| | Best or Good quality links with hardstrip on rhs |
| CROSSQU7 level 2 | Best or Good quality links with no hardstrip on rhs |

This factor was similar to QUAL4, with the higher quality level split according to the presence of an offside hardstrip, and with no significant difference between Good quality links with an offside hardstrip and Lower and Urban quality links.

The other significant variables (at the 5 per cent level) were hilliness (Hilli) and the presence of a safety fence (CRASHF), and these were included in the model. Visibility (represented by its harmonic mean Visib and its maximum value Max_vis) and maximum curvature (minimum radius) Cmaxr were borderline significant, but gave illogical values, so were not included in the model. The best fit model was:

$$\ln(A) = (-9.94 - 0.0225t) + 0.630.\ln(Q) + 0.914.\ln(L_L) + d3.\text{CROSSQU7} + d4.\text{CRASHF} + 0.0202.\text{Hilli},$$

or, in its exponent form,

$$A = 4.82\text{E-}5.e^{-0.0225t}.Q^{0.630}.L_L^{0.914}.\exp(d3.\text{CROSSQU7} + d4.\text{CRASHF} + 0.0202.\text{Hilli}),$$

| | | |
|---|---|---|
| where | $d3 = 0$ | Lower or Urban quality roads, Best or Good quality links with hardstrip. |
| | $d3 = 0.651$ | Best or Good quality links with no hardstrip. |
| and | $d4 = 0$ | No safety fence |
| | $d4 = -0.400$ | Safety fence present |

The flow parameter $\alpha$ was significantly different from 1 (at the 5 per cent level), but the length parameter $\beta$ was not.

The hilliness parameter was 0.0202. Therefore an increase in hilliness of 1 metre/km would increase accidents by 2.0 per cent.

A link with a safety fence present (which had been defined as a link with safety fence along more than half its length) would have 33.0 per cent fewer accidents than a link with no fence present. Evidently the safety fence prevented the vehicles crossing the central reservation.

The factor CROSSQU7 suggested that the presence of hardstrips had an effect on accidents on Best or Good quality roads. The absence of hardstrip on these roads increased accidents by 91.7 per cent. Again, this might be attributable to the speeds attained on roads of this quality as compared to the Lower or Urban quality roads.

### 15.4.5 *Group Y: Multi-vehicle accidents, going ahead on a bend*

There were 47 accidents in the group; the small number made it difficult to produce robust models. Initially, the traffic flow parameter $\alpha$ and length parameter $\beta$ were not significantly different from 0. The flow parameter was therefore constrained to $\alpha = 1$, but the length parameter was left unconstrained, and became significant when other variables were added. The only other significant variables were Propc_d, the proportion with radius of curvature in Range D (at the 1 per cent level), and quality (at the 5 per cent level). For consistency with other groups, quality was represented by the factor QUAL4. The best fit model was:

$$\ln(A) = (-14.99 - 0.0225t) + \ln(Q) + 0.434.\ln(L_L) + 2.315.\text{Propc\_d} + d2.\text{QUAL4},$$

or, in its exponent form,

$$A = 3.09\text{E-}7.e^{-0.0225t}.Q.L_L^{0.434}.\exp(2.315.\text{Propc\_d} + d2.\text{QUAL4}),$$

| | | |
|---|---|---|
| where | $d2 = 0$ | for level 1 of QUAL4 (Best or Good quality) |
| | $d2 = -0.762$ | for level 2 of QUAL4 (Lower or Urban quality) |

The length parameter $\beta$ was significantly different from 1 (at the 0.1 per cent level).

The Lower and Urban quality sites again had the lower accident rate with Best or Good quality roads having more than twice as many accidents.

The parameter value of Propc_d suggested that a link with radii of curvature in range D for a tenth of its length would have 26 per cent more accidents than a link with radii outside this range. This was a large effect, and there were several possible reasons, all of which might occur in some measure.

One was that the presence of relatively sharp curves might actually cause an increase in accidents (though it is worth remembering that range D curves are not especially tight, being curves below 1400m radius).

A second possibility was that the larger the proportion of link which is on bend, the greater the probability that when an accident occurred, for whatever reason, it occurred on a bend. The point is that a model for accidents on bends required a measure of the length of the link that is ‘bend’. This was difficult to define because we do not know what degree of curvature would be treated as a ‘bend’ by the police officers completing the STATS19 forms. However, since such a variable was absent, other variables which were likely to be correlated with it tended to appear in these models. Such variables were bendiness and design speed. When these variables were statistically significant in ‘accident on bend’ models, it was not possible to know how much of the effect was attributable to acting as a proxy for ‘length with bend’ and how much was a residual real effect. It should be noted that the same argument applies for accident groups which included ‘accidents not on a bend’.

In this case, this conclusion on the effect of bends was not very reliable, being based on a model with only 47 accidents and having the traffic flow parameter constrained to 1.

### *15.4.6 Group Z: Multi-vehicle accidents, other manoeuvre*

There were 658 accidents in the group. The most significant variables (at the 1 per cent level) were those relating to the presence of a kerb on the nearside and of a continuous white line on either side - once again illustrating the importance of edge treatment in its various forms. A factor representing combinations of kerb and white lines was formed, but when reduced to statistically distinct combinations it was found to give no improvement in fit over the simple kerb factor KERBF_lhs which indicated the presence or absence of a nearside kerb.

Link quality (at the 1 per cent level) and the proportion of link with a safety fence (Crash_p) (at the 5 per cent level) were also significant. Here, quality was reduced to a new factor QUAL6, where level 1 was Best quality and level 2 was a combination of Good, Urban and Lower quality links.

The best fit model was:

$$\ln(A) = (-13.10 - 0.0225t) + 1.114.\ln(Q) + 0.745.\ln(L_L) + d5.QUAL6 + d6.KERBF\_lhs + 0.208.Crash\_p,$$

or, in its exponent form,

$$A = 2.05E\text{-}6.e^{-0.0225t}.Q^{1.114}.L_L^{0.745}.\exp(d5.QUAL6 + d6.KERBF\_lhs + 0.208.Crash\_p),$$

| | | |
|---|---|---|
| where | d5 = 0 | for level 1 of QUAL6 (Best quality) |
| | d5 = -0.325 | for level 2 of QUAL6 (Good, Lower or Urban quality) |
| and | d6 = 0 | for level 1 of KERBF_lhs (No kerb) |
| | d6 = 0.305 | for level 2 of KERBF_lhs (Kerb present). |

The flow parameter $\alpha$ was not significantly different from 1 but the length parameter $\beta$ was significantly different (at the 0.1 per cent level).

The factor QUAL6 suggested that for this accident group the links of Best quality had the highest accident rate, with the other quality groups having 27.7 per cent fewer accidents.

The kerb factor suggested that kerbs had an adverse effect on accidents, increasing accidents by 35.7 per cent.

The parameter for Crash_p suggested that the higher the proportion of link with a safety fence the higher the accidents. A link with a fence along half of its length would have 11 per cent more accidents than a link with no fence. The explanation is that this accident group encompassed several types of accidents, and might well include accidents where vehicles had rebounded off the safety fences rather than crossed the central reservation (which would put them in Group X).

### *15.4.7 Group S: Multi-vehicle accidents, one vehicle parked*

There were 406 accidents in the group. The most significant variables were (at the 1 per cent level) the number of accesses on the right hand side (Acc_rhs) and (at the 5 per cent level) quality (which reduced to the factor QUAL4 used in earlier groups), down-hilliness (Dnhilli) and the proportion with radius of curvature in range C (Propc_c). These variables were added to the model. The number of accesses appeared to be correlated with link length, which seems reasonable, but replacing it with accesses per km worsened the fit.

The best fit model was:

$$\ln(A) = (-12.99 - 0.0225t) + 1.022.\ln(Q) + 0.752.\ln(L_L) + 0.094.Acc\_rhs + d2.QUAL4 + 0.0167.Dnhilli + 0.613.Propc\_c,$$

or, in its exponent form,

$$A = 2.28E\text{-}6.e^{-0.0225t}.Q^{1.022}.L_L^{0.752}.\exp(0.094.Acc\_rhs + d2.QUAL4 + 0.0167.Dnhilli + 0.613.Propc\_c),$$

| | | |
|---|---|---|
| where | d2 = 0 | for level 1 of QUAL4 (Best or Good quality) |
| | d2 = -0.370 | for level 2 of QUAL4 (Lower or Urban quality). |

The flow parameter $\alpha$ was not significantly different from 1 but the length parameter $\beta$ was significantly different (at the 1 per cent level).

The access parameter of 0.094 suggested that an additional access on the right hand side increased accidents by 9.8 per cent. A parked vehicle might not be clearly visible to vehicles emerging from an access on the right hand side of the carriageway.

Again the QUAL4 factor indicated that the Lower or Urban quality roads were the safest, having 30.9 per cent fewer accidents than the good quality roads. On lower speed roads, evasive action could be more easily taken to avoid parked vehicles.

The down-hilliness parameter suggested that an increase of 1 metre/km for this variable would increase accidents by 1.7 per cent.

The parameter for the variable representing the proportion of link with radius of curvature in range C was 0.613. A link with a tenth of its length with this curvature would have 6.3 per cent more accidents than a link with curvature outside this range.

## 15.5 Level 2 models for single-vehicle accident groups: Accidents by group as function of traffic flow, link length and major design variables

The Level 2 model results for D2 links and single-vehicle accidents (accident groups L to R) are shown in Table 15.3.

### *15.5.1 Group L: Single-vehicle accidents, vehicle left carriageway nearside on bend*

There were 133 accidents in the group. The flow parameter was not significantly different from zero, which suggested that the models were not very robust since the number of accidents was low. A somewhat more robust model was obtained by setting the flow parameter to 1.

The most significant variables were those relating to bendiness and link quality (at the 1 per cent level) and the presence of white edge lines (at the 5 per cent level). Quality reduced to the 2-level factor QUAL4 as used in earlier models.

**Table 15.3 Level 2 models (D2 links) for single-vehicle accident groups: Accidents by group as a function of traffic flow, link length and design variables**

| Model | Model terms | Parameter value | s.e. | Deviance difference | *Multiplicative effect compared to mean at:* Min | Max | *% change in accidents for unit change in variable* Unit | %change | Residual deviance | Degrees of freedom | Scale factor |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Group L: Single-vehicle Accidents, vehicle left carriageway nearside on bend (133)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -3.903 | 0.0860 | | | | | | 472.7 | 517 | 1.672 |
| 2A | ln $k_0$ | -4.44 | 1.90 | | | | | | 427.5 | 515 | 1.358 |
| | α | 0.024 | 0.206 | 0.0 | | | | | | | |
| | β | 0.650 | 0.103 | 43.6 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a1.Bendi + d2.QUAL4] | | | | | | | | | | | |
| Full | ln $k_0$ | -14.30 | 0.216 | | | | | | 396.4 | 514 | 1.555 |
| | α | 1 | 0 | | | | | | | | |
| | β | 0.940 | 0.124 | 69.4 | | | | | | | |
| | Bendi | 0.0219 | 0.0028 | 48.8 | 0.452 | 32.65 | Factor | +25% | | | |
| | QUAL4 | -0.741 | 0.242 | 10.0 | | | Factor | -62% | | | |
| ***Group M: Single-vehicle Accidents, vehicle left carriageway nearside, not on bend (762)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -2.158 | 0.0362 | | | | | | 996.2 | 517 | 2.289 |
| 2A | ln $k_0$ | -7.342 | 0.827 | | | | | | 649.5 | 515 | 1.274 |
| | α | 0.519 | 0.0887 | 34.5 | | | | | | | |
| | β | 0.785 | 0.0455 | 343.3 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[d5.QUAL6 + d7.DGNSPD2] | | | | | | | | | | | |
| Full | ln $k_0$ | -5.59 | 1.01 | | | | | | 628.5 | 513 | 1.229 |
| | α | 0.553 | 0.0889 | 39.2 | | | | | | | |
| | β | 0.765 | 0.0474 | 306.2 | | | | | | | |
| | QUAL6 | -0.279 | 0.0769 | 13.3 | | | Factor | -24% | | | |
| | DGNSPD2 | -0.965 | 0.313 | 7.3 | | | 120:100 | -62% | | | |
| **Group N: Single-vehicle Accidents, vehicle left carriageway offside on bend (65)** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -4.619 | 0.124 | | | | | | 305.3 | 517 | 1.356 |
| 2A | ln $k_0$ | -12.74 | 2.81 | | | | | | 283.6 | 515 | 1.345 |
| | α | 0.852 | 0.300 | 8.1 | | | | | | | |
| | β | 0.596 | 0.148 | 17.9 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a1.Bendi + a2.Hilli] | | | | | | | | | | | |
| Full | ln $k_0$ | -13.94 | 2.68 | | | | | | 240.4 | 513 | 1.279 |
| | α | 0.812 | 0.282 | 8.2 | | | | | | | |
| | β | 0.991 | 0.171 | 39.1 | | | | | | | |
| | Bendi | 0.0188 | 0.0037 | 21.2 | 0.51 | 19.92 | 10 deg/km | 21% | | | |
| | Hilli | 0.0346 | 0.0125 | 7.3 | 0.62 | 3.54 | 1 m/km | 3.5% | | | |
| **Group O: Single-vehicle Accidents, vehicle left carriageway offside, not on bend (425)** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -2.741 | 0.0484 | | | | | | 831.9 | 517 | 2.028 |
| 2A | ln $k_0$ | -8.45 | 1.11 | | | | | | 619.1 | 515 | 1.447 |
| | α | 0.573 | 0.119 | 23.3 | | | | | | | |
| | β | 0.829 | 0.0615 | 210.2 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[d1.CROSS8 + d5.QUAL6] | | | | | | | | | | | |
| Full | ln $k_0$ | -8.08 | 1.12 | | | | | | 579.6 | 513 | 1.297 |
| | α | 0.525 | 0.121 | 18.9 | | | | | | | |
| | β | 0.860 | 0.0674 | 199.7 | | | | | | | |
| | CROSS8 | 0.542 | 0.106 | 25.8 | | | Factor | +72% | | | |
| | QUAL6 | -0.457 | 0.106 | 19.1 | | | Factor | -37% | | | |

**Table 15.3 Level 2 models (D2 links) for single-vehicle accident groups: Accidents by group as a function of traffic flow, link length and design variables (continued)**

| | | | | | *Multiplicative effect compared to mean at:* | | *% change in accidents for unit change in variable* | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| *Model* | *Model terms* | *Parameter value* | *s.e.* | *Deviance difference* | *Min* | *Max* | *Unit* | *%change* | *Residual deviance* | *Degrees of freedom* | *Scale factor* |
| **Group R: Single-vehicle Accidents, vehicle did not leave carriageway (345)** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | $\ln k_0$ | -2.950 | 0.0537 | | | | | | 723.7 | 517 | 1.777 |
| 2A | $\ln k_0$ | -8.85 | 1.23 | | | | | | 574.5 | 515 | 1.387 |
| | $\alpha$ | 0.599 | 0.131 | 20.9 | | | | | | | |
| | $\beta$ | 0.757 | 0.0670 | 145.4 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[d8.QUAL8 + d9.KERB1 + a3.Obstrp\_rhs + a1.Bendi]$ | | | | | | | | | | | |
| Full | $\ln k_0$ | -6.78 | 1.23 | | | | | | 515.0 | 511 | 1.201 |
| | $\alpha$ | 0.315 | 0.133 | 5.6 | | | | | | | |
| | $\beta$ | 0.897 | 0.0740 | 172.8 | | | | | | | |
| | QUAL8 | 0.584 | 0.155 | 13.1 | | | Factor | 79% | | | |
| | KERB1 | 0.484 | 0.128 | 13.6 | | | Factor | 62% | | | |
| | Obstrp_rhs | 2.579 | 0.818 | 7.6 | 0.99 | 3.32 | 10% | +29% | | | |
| | Bendi | 0.00598 | 0.00227 | 6.4 | 0.81 | 2.59 | 10 deg/km | +6.2% | | | |

The white line parameters were very variable according to which other variables were included, and gave counter-intuitive results. It was therefore decided that their inclusion did not represent a real effect, so they were excluded.

The best fit model was:

$$\ln(A) = (-14.30 - 0.0225t) + \ln(Q) + 0.940.\ln(L_L) + 0.0219.Bendi + d2.QUAL4,$$

or, in its exponent form,

$$A = 6.16E\text{-}7.e^{-0.0225t}.Q.L_L^{0.940}.\exp(0.0219.Bendi + d2.QUAL4),$$

where d2 = 0 for level 1 of QUAL4 (Best or Good quality)

d2 = -0.741 for level 2 of QUAL4 (Lower or Urban quality).

The length parameter $\beta$ was not significantly different from 1.

The model suggested that an increase in bendiness of 10 deg/km would increase accidents by 24.5 per cent. This was a large effect, for which the alternative explanations discussed under Group Y would apply. In addition, the traffic flow parameter was set to 1 in order to produce a more robust model, so the results could not be very reliable. However, the Group L results were similar to those from Group N, which suggested the possibility that the effect of bendiness could be real.

The Lower or Urban quality links were safer than the Best or Good quality links, having less than half as many accidents. As with other accident groups this may be related to the speeds attained on the links and the driver's control of the car on bends at such speeds.

### *15.5.2 Group M: Single-vehicle accidents, vehicle left carriageway nearside, not on bend*

There were 762 accidents in the group. The most significant variables were link quality (at the 1 per cent level) and the number of accesses on the nearside (Acc_lhs) and design speed (DGNSPEED) (at the 5 per cent level). Quality was reduced to a two level factor, QUAL6 where level 1 was the Best quality roads and level 2 was the Good, Lower or Urban quality roads. This factor was used earlier in the model for accident group Z.

The four level factor for design speed was reduced to two levels, DGNSP2, where level 1 represented design speed of 100A and 100B and level 2 represented 120A and 120B. The inclusion of design speed also brought the speed variables COBA-9 and COBA-10 speed formulae into significance (at the 5 per cent level). However their formulae were similar to $V_{50,wet}$ from which DGNSPEED was derived, and since speed terms are highly correlated only one can be included in a model.

The best fit model was:

$$\ln(A) = (-5.59 - 0.0225t) + 0.553.\ln(Q) + 0.765.\ln(L_L) + d5.QUAL6 + d7.DGNSP2,$$

or, in its exponent form,

$$A = 3.74E\text{-}3.e^{-0.0225t}.Q^{0.553}.L_L^{0.765}.\exp(d5.QUAL6 + d7.DGNSP2),$$

where d5 = 0 for level 1 of QUAL6 (Best quality)

d5 = -0.279 for level 2 of QUAL6 (Good, Lower or Urban quality)

and d7 = 0 for Design speed of 100A or 100B.

d7 = -0.965 for Design speed of 120A or 120B.

The flow parameter $\alpha$ was significantly different from 1 (at the 0.1 per cent level), as was the length parameter $\beta$ (at the 0.1 per cent level).

Best quality schemes had no major junctions and hence the speeds attained would be high compared to Good, Lower or Urban quality schemes with major junctions. These three quality groups had 24.4 per cent fewer accidents than the Best quality roads.

The model suggested that a link with design speed 100 would have more than 2½ times as many accidents as a link with design speed 120. Design speed was derived from $V_{50,wet}$ which was calculated using bendiness, down-hilliness and traffic flow. Traffic flow was already in the model, and the other variables were tested in the model, but Design speed was found to be more significant, though only at the 5 per cent level. A high design speed reflected a straight road, implying that bendy roads would have more accidents, but other explanations, as discussed under Group Y, are possible.

#### *15.5.3 Group N: Single-vehicle accidents, vehicle left carriageway offside on bend*

There were 65 accidents in the group; the small number made it difficult to produce robust models. The most significant variables were bendiness (at the 1 per cent level) and hilliness (at the 5 per cent level). The 4-level factor Quality was also marginally significant at the 5 per cent level, but its four levels were not significantly different, so it was decided to exclude the factor altogether.

The best fit model was:

$$\ln(A) = (-13.94 - 0.0225t) + 0.812.\ln(Q) + 0.991.\ln(L_L) + 0.0188.\text{Bendi} + 0.0346.\text{Hilli},$$

or, in its exponent form,

$$A = 8.83E\text{-}7.e^{-0.0225t}.Q^{0.812}.L_L^{0.991}.\exp(0.0188.\text{Bendi} + 0.0346.\text{Hilli}),$$

Neither the flow parameter $\alpha$ nor the length parameter $\beta$ was significantly different from 1.

The model suggested that an increase in bendiness of 10 deg/km would increase accidents by 20.7 per cent, but other explanations, as discussed under Group Y, are possible. An increase in hilliness of 1 metre/km would increase accidents by 3.5 per cent.

#### *15.5.4 Group O: Single-vehicle accidents, vehicle left carriageway offside, not on bend*

There were 425 accidents in the group. The most significant variables (at the 1 per cent level) were various relating to kerbs and hardstrips. A factor was developed to test the various combinations of hardstrips and kerbs, and it was reduced to the 2-level factor CROSS8 which had been included in the model for overtaking accidents (Group T). Quality also appeared as significant (at the 1 per cent level) and was reduced to a 2 level factor, QUAL6, which had been used in the models for accident groups M and Z. With the addition of CROSS8 and QUAL6, visibility and median verge width became significant (at the 5 per cent level). However, Vwid_rhs had been excluded from previous models due to problems with its interpretation, and visibility was not included as it was only marginally significant.

The best fit model was:

$$\ln(A) = (-8.080 - 0.0225t) + 0.525.\ln(Q) + 0.860.\ln(L_L) + d1.\text{CROSS8} + d5.\text{QUAL6},$$

or, in its exponent form,

$$A = 3.10E\text{-}4.e^{-0.0225t}.Q^{0.525}.L_L^{0.860}.\exp(d1.\text{CROSS8} + d5.\text{QUAL6}),$$

| | | | |
|---|---|---|---|
| where | d1 = 0 | for level 1 of CROSS8 (Any kerbs set-back behind hardstrips) |
| | d1 = 0.542 | for level 2 of CROSS8 (Kerbs at carriageway edge) |
| and | d5 = 0 | for level 1 of QUAL6 (Best quality) |
| | d5 = -0.457 | for level 2 of QUAL6 (Good, Lower or Urban quality) |

The flow parameter $\alpha$ was significantly different from 1 (at the 0.1 per cent level), but the length parameter $\beta$ was not.

A link where the carriageway edge was level or with kerbs which were not set back would have 71.9 per cent more accidents than a link where any kerbs present were set back behind hardstrips.

The Best quality schemes had no major junctions and hence the speeds attained on such schemes would be high. The model suggested that schemes of Good, Lower or Urban quality links would have 36.7 per cent fewer accidents than links on Best quality schemes.

#### *15.5.5 Group R: Single-vehicle accidents, vehicle did not leave carriageway*

There were 345 accidents in the group.

The most significant variable was link quality (at the 1 per cent level). This was reduced to a new 2-level factor QUAL8 where level 1 included Best, Good and Lower quality schemes and level 2 included Urban quality schemes.

The next most significant variables (at the 1 per cent level) were the continuous white line and kerb variables, another example of the importance of edge treatments. These variables were correlated and hence it was decided to combine them into one factor, which was reduced to a two level factor, KERB1, defined as follows:

| | |
|---|---|
| KERB1 Level 1: | White lines on both sides. |
| | White lines on one side without kerbs. |
| | White lines on one side with kerbs on same side. |
| KERB1 Level 2: | Kerbs only. |
| | No edge treatment. |
| | White lines on one side of the road with kerb on opposite side. |
| | White lines on one side of the road with kerb on both sides. |

The two level factor KERB1 therefore represented the various combinations of continuous white line and kerb on the right and left hand side of the carriageway. Level 1 represented links where any kerbs present were set back behind continuous white lines. In contrast, level 2 represented links with kerbs which were not set back behind white lines or where the carriageway edge was level. KERB1 was therefore similar to the factor CROSS8 but with the substitution of continuous white lines for hardstrips.

Open_yr and Vwid_lhs were found to be significant (at the 5 per cent level), but these were not included due to problems with interpretation; Obstrp_rhs was added in place of verge width as in previous models. The bendiness variable was also significant (at the 5 per cent level) and was added to the model, giving the best fit model as:

$$\ln(A) = (-6.780 - 0.0225t) + 0.315.\ln(Q) + 0.897.\ln(L_L) + d8.QUAL8 + d9.KERB1 + 2.579.Obstrp\_rhs + 0.00598.Bendi,$$

or, in its exponent form,

$$A = 1.14E\text{-}3.e^{-0.0225t}.Q^{0.315}.L_L^{0.897}.\exp(d8.QUAL8 + d9.KERB1 + 2.579.Obstrp\_rhs + 0.00598.Bendi),$$

| | | |
|---|---|---|
| where | d8 = 0 | for level 1 of QUAL8 (Best, Good or Lower quality roads), |
| | d8 = 0.584 | for level 2 of QUAL8 (Urban quality), |
| and | d9 = 0 | for level 1 of KERB1 (no kerbs, or marked by white lines), |
| | d9 = 0.484 | for level 2 of KERB1 (unmarked kerbs or no edge treatment). |

The flow parameter $\alpha$ was significantly different from 1 (at the 0.1 per cent level), but the length parameter $\beta$ was not.

This accident group was a catch-all group combining all single-vehicle accidents in which the vehicle did not leave the carriageway. It therefore included overtaking, changing lane and other manoeuvres, causing difficulty in interpreting the results.

QUAL8 had not been used in earlier models. It suggested that for this accident group the Urban quality roads had 79.3 per cent more accidents.

The factor KERB1 suggested that if kerbs were present, white lines should be painted alongside to emphasise their presence. Roads with no edge treatment or unmarked kerbs increased accidents by 62.3 per cent.

A link with a continuous obstruction along a tenth of its length on the right hand side would have 29.4 per cent more accidents than a link with no obstructions. This was consistent with the overtaking multi-vehicle accidents.

An increase in bendiness of 10 deg/km would increase accidents by 6.2 per cent. This was surprising, because on bendy roads more vehicles might be expected to leave the road, but this group contained accidents where the vehicle did not leave the road. As stated above, however, this group was a catch-all group, and may have contained accidents which should have been allocated to other groups when the STATS19 records were coded.

## 15.6 Level 2 models for other accident groups: Accidents by group as function of traffic flow, link length and major design variables

The Level 2 model results for D2 links and other accident groups (accident groups J to K) are shown in Table 15.4.

### *15.6.1 Group J: Accidents involving pedestrians*

There were 228 accidents in the group.

The most significant variables (at the 1 per cent level) were those relating to edge treatment, that is, kerbs, hardstrips and continuous white lines. An 11-level factor for various combinations of kerbs and hardstrips was tested (continuous white lines were not included as they are correlated with hardstrips), and this was gradually reduced to a 2-level factor, where level 1 was the presence of a hardstrip one or both sides, and level 2 was kerbs without hardstrips or no edge treatment. However, in view of the fact that very few links had hardstrips only on the right hand side, this factor was essentially the same as the basic factor HARDF_lhs, the presence of a hardstrip on the left hand side. This variable was therefore used in the model.

The other significant variables (at the 5 per cent level) were the proportion of link with a steep gradient (of more than |4| per cent - Propg_4), and maximum visibility (Max_vis), which indicated an increase in accidents on steep gradients and a reduction with greater visibility, which seems reasonable.

The best fit model was:

$$\ln(A) = (-10.71 - 0.0225t) + 0.902.\ln(Q) + 0.643.\ln(L_L) + d10.Hardf\_lhs + 1.083.Propg\_4 - 0.00221.Max\_vis,$$

or, in its exponent form,

$$A = 2.23E\text{-}5.e^{-0.0225t}.Q^{0.902}.L_L^{0.643}.\exp(d10.Hardf\_lhs + 1.083.Propg\_4 - 0.00221.Max\_vis),$$

| | | |
|---|---|---|
| where | d10 = 0 | for level 1 of HARDF_lhs (No hardstrip on the left hand side), |
| | d10 = -0.589 | for level 2 of HARDF_lhs (Hardstrip present on left hand side). |

The flow parameter $\alpha$ was not significantly different from 1 but the length parameter $\beta$ was significantly different (at the 0.1 per cent level).

The presence of a hardstrip was associated with a decrease in accidents of 44.5 per cent.The significance of the hardstrip factor might suggest that pedestrians on dual carriageways were safer where there was a hardstrip on the left hand side. However, it was more likely to result from the fact that more dual carriageways with hardstrips are found in rural areas where there are fewer pedestrians anyway.

A link with gradient greater than |4| per cent for a tenth of its length would have 11.4 per cent more accidents than a link with gradient outside this range. Steep gradients might have an effect on drivers control, visibility and their stopping distance.

Reducing the maximum visibility by 100m would increase accidents by 24.7 per cent. Evidently, increased visibility would increase the drivers’ advance warning of pedestrians on the carriageway.

### *15.6.2 Group K: Accidents at accesses*

There were 85 accidents in the group. Some of the variables in the model had extreme effects compared to other accident group models, which suggested that with a low number of accidents, the model was not very robust.

The length parameter was not significantly different from zero, suggesting that access accidents were independent of link length. Accidents were, however, dependent on the number of accesses on the left hand side (the most significant variable), which would tend to be

**Table 15.4 Level 2 models (D2 links) for other accident groups: Accidents by group as a function of traffic flow, link length and design variables**

| Model | Model terms | Parameter value | s.e | Deviance difference | *Multiplicative effect compared to mean at:* Min | Max | *% change in accidents for unit change in variable* Unit | %change | Residual deviance | Degrees of freedom | Scale factor |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Group J: Accidents involving pedestrians (228)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -3.364 | 0.0660 | | | | | | 584.8 | 517 | 1.492 |
| 2A | ln $k_0$ | -13.51 | 1.48 | | | | | | 520.4 | 515 | 1.299 |
| | $\alpha$ | 1.080 | 0.157 | 47.1 | | | | | | | |
| | $\beta$ | 0.409 | 0.0755 | 31.4 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[d10.HARDF_lhs + a10.Propg_4 + a11.Max_vis] | | | | | | | | | | | |
| Full | ln $k_0$ | -10.71 | 1.655 | | | | | | 489.2 | 512 | 1.287 |
| | $\alpha$ | 0.902 | 0.159 | 32.5 | | | | | | | |
| | $\beta$ | 0.643 | 0.0934 | 52.4 | | | | | | | |
| | HARDF_lhs | -0.589 | 0.144 | 16.6 | | | Factor | -45% | | | |
| | Propg_4 | 1.083 | 0.405 | 6.2 | 0.95 | 2.79 | 10% | +11% | | | |
| | Max_vis | -0.00221 | 0.000953 | 5.1 | 2.18 | 0.90 | -100m | +25% | | | |
| ***Group K: Accidents at accesses (85)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -4.350 | 0.106 | | | | | | 429.5 | 517 | 3.126 |
| 2A | ln $k_0$ | -14.61 | 2.36 | | | | | | 410.1 | 515 | 2.681 |
| | $\alpha$ | 1.104 | 0.251 | 19.4 | | | | | | | |
| | $\beta$ | 0.089 | 0.113 | 0.6 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$.exp[a12.Acc_lhs + d11.QUAL9 + d4.CRASH_f + d12.KERBS] | | | | | | | | | | | |
| Full | ln $k_0$ | -14.14 | 2.51 | | | | | | 337.1 | 511 | 2.001 |
| | $\alpha$ | 0.945 | 0.269 | 12.5 | | | | | | | |
| | $\beta$ | -0.217 | 0.133 | 2.5 | | | | | | | |
| | Acc_lhs | 0.229 | 0.0368 | 30.4 | | | 1 access | +26% | | | |
| | QUAL9 | 0.979 | 0.250 | 13.3 | | | Factor | 166% | | | |
| | CRASH_f | -0.846 | 0.261 | 11.6 | | | Factor | -57% | | | |
| | KERBS | 0.804 | 0.301 | 8.1 | | | Factor | 123% | | | |

larger on longer links, so the insignificance of the length parameter was understandable.

Link quality was found to be significant (at the 1 per cent level), and reduced to a 2-level factor QUAL9 where Good quality links had a higher rate than other qualities.

Edge treatment was found to be significant (at the 5 per cent level), and a multi-level factor was reduced to a 2-level factor KERBS indicating whether a kerb was present on either or both sides. The presence of a safety fence (CRASH_f) was significant (at the 5 per cent level) and was included. Minimum visibility, verge width and maximum curvature were only marginally significant (at the 5 per cent level), so these variables were not included in the model.

The best fit model was:

$$\ln(A) = (-14.14 - 0.0225t) + 0.945.\ln(Q) - 0.217.\ln(L_L) + 0.229.\text{Acc\_lhs} + \text{d11.QUAL9} + \text{d4.CRASH\_f} + \text{d12.KERBS},$$

or, in its exponent form,

$$A = 7.23E\text{-}7.e^{-0.0225t}.Q^{0.945}.L_L^{0.217}.\exp(0.229.\text{Acc\_lhs} + \text{d11.QUAL9} + \text{d4.CRASH\_f} + \text{d12.KERBS}),$$

where d4 = 0 for level 1 of CRASH_f (No safety fence),

d4 = -0.846 for level 2 of CRASH_f (Safety fence present),

d11 = 0 for level 1 of QUAL9 (Best, Lower or Urban quality).

d11 = 0.979 for level 2 of QUAL9 (Good quality),

d12 = 0 for level 1 of KERBS (No kerbs on either side),

d12 = 0.804 for level 2 of KERBS (Kerbs present on either side).

The flow parameter $\alpha$ was not significantly different from 1 but the length parameter $\beta$ was significantly different (at the 0.1 per cent level). The length parameter was not significantly different from zero.

An additional access on the left hand side would increase accidents by 25.7 per cent.

The quality factor QUAL9 had not been included in any earlier accident group model. It suggested that Good quality schemes had the highest accident rate with over two and a half as many accidents. Good quality roads were high speed roads but with some roundabouts. It was possible that accesses on Good quality links were used more than on Best quality links.

Links with safety fences would have less than half as many accidents as those without.

The presence of kerbs on either side of the carriageway would more than double the number of accidents.

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

Level 2 models were also developed for the accident classes defined in Section 5.5, that is, Class 1 (single-vehicle accidents), Class 2 (accidents involving 2 vehicles) and Class 3 (accidents involving 3 or more vehicles). The Level 2 model results for D2 links and accident classes 1 to 3 are shown in Table 15.5.

### *15.7.1 Class 1: Accidents involving a single vehicle*

There were 1731 accidents in the class.

The most significant variable was link quality (at the 1 per cent level). This was reduced to a new 2-level factor QUAL11 where level 1 included Best and Urban quality

**Table 15.5 Level 2 models (D2 links): Accidents by accident class as a function of traffic flow, link length and design variables**

| | | | | | *Multiplicative effect compared to mean at:* | | *% change in accidents for unit change in variable* | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| *Model* | *Model terms* | *Parameter value* | *s.e.* | *Deviance difference* | *Min* | *Max* | *Unit* | *%change* | *Residual deviance* | *Degrees of freedom* | *Scale factor* |
| ***Class 1: Accidents involving a single vehicle (1731)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -1.337 | 0.0240 | | | | | | 1687.0 | 517 | 3.693 |
| 2A | ln $k_0$ | -6.538 | 0.549 | | | | | | 921.8 | 515 | 1.833 |
| | $\alpha$ | 0.522 | 0.0588 | 79.0 | | | | | | | |
| | $\beta$ | 0.772 | 0.0301 | 757.0 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[d13.QUAL11 + a1.Bendi + d1.CROSS8]$ | | | | | | | | | | | |
| Full | ln $k_0$ | -5.858 | 0.543 | | | | | | 851.9 | 512 | 1.655 |
| | $\alpha$ | 0.428 | 0.0587 | 53.7 | | | | | | | |
| | $\beta$ | 0.820 | 0.0337 | 724 | | | | | | | |
| | QUAL11 | -0.297 | 0.0531 | 32.3 | | | Factor | -26% | | | |
| | Bendi | 0.00486 | 0.00103 | 20.9 | 0.84 | 2.17 | 10 deg/km | +5.0% | | | |
| | CROSS8 | 0.232 | 0.0527 | 19.2 | 19.2 | | Factor | +26% | | | |
| ***Class 2: Accidents involving 2 vehicles (1648)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -1.386 | 0.0246 | | | | | | 1611.0 | 517 | 3.356 |
| 2A | ln $k_0$ | -9.653 | 0.564 | | | | | | 956.8 | 515 | 1.957 |
| | $\alpha$ | 0.862 | 0.0601 | 208 | | | | | | | |
| | $\beta$ | 0.677 | 0.0302 | 569 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[d2.QUAL4 + d1.CROSS8 + a1.Bendi + a3.Obstrp\_rhs]$ | | | | | | | | | | | |
| Full | ln $k_0$ | -9.589 | 0.574 | | | | | | 903.1 | 511 | 1.854 |
| | $\alpha$ | 0.839 | 0.0621 | 185 | | | | | | | |
| | $\beta$ | 0.679 | 0.0342 | 467 | | | | | | | |
| | QUAL4 | -0.336 | 0.0645 | 27.9 | | | Factor | -29% | | | |
| | CROSS8 | 0.247 | 0.0553 | 19.9 | | | Factor | +28% | | | |
| | Bendi | 0.00373 | 0.00108 | 11.4 | 0.87 | 1.81 | 10 deg/km | +3.8% | | | |
| | Obstrp_rhs | 1.466 | 0.490 | 7.6 | 0.99 | 1.98 | 10% | +16% | | | |
| ***Class 3: Accidents involving 3 or more vehicles (590)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | ln $k_0$ | -2.413 | 0.0411 | | | | | | 1032.0 | 517 | 2.473 |
| 2A | ln $k_0$ | -14.294 | 0.941 | | | | | | 768.3 | 515 | 1.785 |
| | $\alpha$ | 1.255 | 0.0995 | 159.6 | | | | | | | |
| | $\beta$ | 0.605 | 0.0497 | 165.9 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[d14.KERBF\_rhs + d2.QUAL4 + a3.Obstrp\_rhs + a2.Hilli + a4.Acc\_rhs]$ | | | | | | | | | | | |
| Full | ln $k_0$ | -14.493 | 0.983 | | | | | | 712.8 | 510 | 1.564 |
| | $\alpha$ | 1.229 | 0.105 | 139.0 | | | | | | | |
| | $\beta$ | 0.586 | 0.0584 | 110.8 | | | | | | | |
| | KERBF_rhs | 0.498 | 0.0938 | 27.9 | | | Factor | +64% | | | |
| | QUAL4 | -0.343 | 0.103 | 11.3 | | | Factor | -29% | | | |
| | Obstrp_rhs | 2.712 | 0.728 | 9.7 | 0.99 | 3.53 | 10% | +31% | | | |
| | Hilli | 0.0143 | 0.00469 | 8.9 | 0.82 | 1.68 | 1 m/km | +1.4% | | | |
| | Acc_rhs | 0.0752 | 0.0266 | 7.4 | 0.93 | 2.12 | 1 access | +7.8% | | | |

schemes and level 2 included Good and Lower quality schemes. This factor had not been included in the accident group models. However, in several of these models the Urban quality was not significantly different from the Best quality, so the use of QUAL11 with Urban and Best quality combined was consistent with the accident group models.

The next most significant variables were bendiness, which was significant at the 1 per cent level and was included in the model, and various variables relating to hardstrips (some at the 1 per cent level, some at the 5 per cent level). Because kerb variables were also bordering on significance at the 5 per cent level, a factor combining kerbs and hardstrips was developed which reduced to CROSS8 (kerbs set-back or not) as used in earlier models. This was included.

Visibility, design speed and the proportion with radius of curvature in range D were significant (at the 5 per cent level), but were not included due to their low significance and their correlation with bendiness.

The best fit model was:

$$\ln(A) = (-5.858 - 0.0225t) + 0.428.\ln(Q) + 0.820.\ln(L_L) + d13.QUAL11 + 0.00486.Bendi + d1.CROSS8,$$

or, in its exponent form,

$$A = 2.86E\text{-}3.e^{-0.0225t}.Q^{0.428}.L_L^{0.820}.\exp(d13.QUAL11 + 0.00486.Bendi + d1.CROSS8),$$

| | | |
|---|---|---|
| where | d13 = 0 | for level 1 of QUAL11 (Best and Urban quality), |
| | d13 = -0.297 | for level 2 of QUAL11 (Good and Lower quality), |
| and | d1 = 0 | for level 1 of CROSS8 (Any kerbs set-back behind hardstrips) |
| | d1 = 0.232 | for level 2 of CROSS8 (Kerbs at carriageway edge) |

The flow parameter $\alpha$ was significantly different from 1 (at the 0.1 per cent level), as was the length parameter $\beta$ (at the 0.1 per cent level).

For single-vehicle accidents, the Good and Lower quality links had 25.7 per cent fewer accidents than the Best and Urban quality links - an anomalous result because it appears that the extremes of the quality range were more similar to each other than to the middle.

An increase in bendiness of 10 deg/km would increase accidents by 5.0 per cent. The value of the parameter was similar to that found for accident group R, where vehicles did not leave the carriageway.

The parameter for CROSS8 was similar to that found in the level 1 model. It suggested that links with kerbs which were not set back behind a hardstrip would have 26.1 per cent more accidents.

#### *15.7.2 Class 2: Accidents involving two vehicles*

There were 1648 accidents in the class.

The most significant variable was link quality (at the 1 per cent level). This was reduced to the 2-level factor QUAL4 as used in previous models, where level 1 included Best and Good quality schemes and level 2 included Urban and Lower quality schemes.

The next most significant variables were various variables relating to kerbs, hardstrips and white lines. The factor CROSS8 (kerbs set-back or not) was included (significant at the 1 per cent level). There was some correlation with QUAL4 but introducing interactions did not improve the fit.

Bendiness (significant at the 1 per cent level) and the proportion of obstructions on the right hand side (significant at the 5 per cent level) were also included. The proportion of hardstrip on the right hand side was significant (at the 5 per cent level) but was not included due to correlation with CROSS8.

The best fit model was:

$$\ln(A) = (-9.589 - 0.0225t) + 0.839.\ln(Q) + 0.679.\ln(L_L) + d2.QUAL4 + d1.CROSS8 + 0.00373.Bendi + 1.466.Obstrp\_rhs,$$

or, in its exponent form,

$$A = 6.85E\text{-}5.e^{-0.0225t}.Q^{0.839}.L_L^{0.679}.\exp(d2.QUAL4 + d1.CROSS8 + 0.00373.Bendi + 1.466.Obstrp\_rhs),$$

| | | |
|---|---|---|
| where | d1 = 0 | for level 1 of CROSS8 (Any kerbs set-back behind hardstrips) |
| | d1 = 0.247 | for level 2 of CROSS8 (Kerbs at carriageway edge) |
| and | d2 = 0 | for level 1 of QUAL4 (Best or Good quality) |
| | d2 = -0.336 | for level 2 of QUAL4 (Lower or Urban quality). |

The flow parameter $\alpha$ was significantly different from 1 (at the 5 per cent level), as was the length parameter $\beta$ (at the 0.1 per cent level).

The parameters in this model were similar to the level 1 model. Lower or Urban quality links had 28.6 per cent fewer accidents than links on Best or Good quality roads. Links where the carriageway edge was level or with kerbs which were not set back would have 28.1 per cent more accidents than links where any kerbs present were set back behind hardstrips. An increase in bendiness of 10 deg/km will increase accidents by 3.8 per cent. A link with a continuous obstruction along a tenth of its length would have 15.8 per cent more accidents than a link with no obstruction.

#### *15.7.3 Class 3: Accidents involving three or more vehicles*

There were 590 accidents in the class.

The most significant (at the 1 per cent level) variables were various variables relating to kerbs on the right hand side, hardstrips and white lines. Since the white lines and hardstrips were correlated, a factor CROSS10B was developed which combined hardstrips and kerbs. In previous models this had been reduced to CROSS8. For this accident group this was not possible, but after investigation it was decided to include the simple factor Kerbf_rhs relating to kerbs on the right hand side in the model since this was easy to interpret and of comparable significance to the other developed factors.

The next significant variable was link quality (at the 1 per cent level). This was reduced to the 2-level factor QUAL4 as used in previous models, where level 1 included Best and Good quality schemes and level 2 included Urban and Lower quality schemes. Hilliness, and the proportion of obstructions and number of accesses on the right hand side, were significant (at the 5 per cent level) and were included.

The best fit model was therefore:

$$\ln(A) = (-14.493 - 0.0225t) + 1.229.\ln(Q) + 0.586.\ln(L_L) + d14.KERBF\_rhs + d2.QUAL4 + 2.712.Obstrp\_rhs + 0.0143.Hilli + 0.0752.Acc\_rhs,$$

or, in its exponent form,

$$A = 5.08E\text{-}7.e^{-0.0225t}.Q^{1.229}.L_L^{0.586}.\exp(d14.KERBF\_rhs + d2.QUAL4 + 2.712.Obstrp\_rhs + 0.0143.Hilli + 0.0752.Acc\_rhs),$$

where $d14 = 0$ for level 1 of factor KERBF_rhs (Kerb absent),
$d14 = 0.498$ for level 2 of factor KERBF_rhs (Kerb present),

and $d2 = 0$ for level 1 of QUAL4 (Best or Good quality)
$d2 = -0.343$ for level 2 of QUAL4 (Lower or Urban quality).

The flow parameter α was not significantly different from 1 but the length parameter β was significantly different (at the 0.1 per cent level).

The parameter values were similar to those in other models. The presence of the kerb increased accidents by 64.5 per cent. Lower or Urban quality schemes had 29.0 per cent fewer accidents than Best or Good quality schemes. A link with a continuous obstruction along a tenth of its length would have 31.2 per cent more accidents than a link with no obstruction. A increase in hilliness of 1 metre/km increased accidents by 1.4 per cent. An additional access on the right hand side increased accidents by 7.8 per cent.

# 16 Regression analysis results for dual-3 links

This section presents the detailed results from the regression analysis models for Dual-3 links.

There was only a small number (36) of 3-lane dual-carriageway (D3) links, so it was not possible to carry out the full range of accident group models as for the 2-lane (D2) links. The models tested were restricted to Level 1 models plus Level 2 models for the accident classes, where there were expected to be sufficient accidents. Even so, the models obtained were found to be not very robust, so it is recommended that the results obtained should be used only as indicators of the effects of various features on accidents. There were 1024 accidents on the 36 D3 links.

As with the D2 models, the tables give the value of $k$ as $k_0$. For years other than 1990, this must be multiplied by $\exp(\theta t)$, where $t$ is the year relative to 1990. For D3 links, the value of θ to be used is -0.0183, representing a decrease in accident risk of 1.83 per cent per annum. In the logarithmic models, the value of $\ln(k)$ should be taken as $(\ln(k_0) - 0.0183t)$, where $\ln(k_0)$ has the value given in the Tables.

## 16.1 Level 1A models for D3 links

Level 1A results for D3 links are presented in Table 16.1.

The value of the flow term parameter α was 1.031, and was not significantly different from 1. However, in most studies of this type there is a non-linear relationship between accidents and flow, so the parameter was not set to 1 but was left unconstrained in the model.

The length parameter β was significantly different from 1, implying, as for the D2 links, a non-linear relationship between accidents and link length. The value for D3 links, 0.623, was not significantly different from that for D2 links.

The best fit model at Level 1A was therefore:

$$\ln(A) = (-10.16 - 0.0183t) + 1.031.\ln(Q) + 0.623.\ln(L_L),$$

or, in its exponent form,

$$A = 3.87E\text{-}5.e^{-0.0183t}.Q^{1.031}.L_L^{0.623}.$$

The flow parameter α was not significantly different from 1 but the length parameter β was significantly different (at the 0.1 per cent level).

## 16.2 Level 1B models for D3 links

Level 1B models were derived by adding to the Level 1A model deduced above the major design variables listed in Section 14.6. The results are included in Table 16.1.

As for D2 links, variables relating to the presence of kerbs and hardstrips were found to be highly significant (at the 0.1 per cent level). In the D3 case, it was found that the inclusion of one edge treatment variable, the factor for the presence of a hardstrip on the offside, was sufficient. No other edge treatment variables, either singly or in combination, produced a significant improvement over a model containing this factor alone.

Minimum visibility was the only other variable which was significant at the 5 per cent level, giving as the best fit model:

$$\ln(A) = (-4.19 - 0.0183t) + 0.529.\ln(Q) + 0.624.\ln(L_L) + d15.HARDF\_rhs - 0.00185.Min\_vis,$$

or, in its exponent form,

$$A = 0.0151.e^{-0.0183t}.Q^{0.529}.L_L^{0.624}.\exp(d15.HARDF\_rhs - 0.00185.Min\_vis),$$

where $d15 = 0$ for level 1 of factor HARDF_rhs (No offside hardstrip),
$d15 = -0.577$ for level 2 of factor HARDF_rhs (Offside hardstrip present).

The flow parameter α was not significantly different from 1 but the length parameter β was significantly different (at the 0.1 per cent level).

The inclusion of geometric variables at Level 1B altered the flow parameter to 0.529 from its Level 1A value of

**Table 16.1 Level 1 models (D3 links): Total accidents as a function of traffic flow, link length and design variables**

| *Model* | *Model terms* | *Parameter value* | *s.e.* | *Deviance difference* | *Multiplicative effect compared to mean at:* *Min* | *Max* | *% change in accidents for unit change in variable* *Unit* | *%change* | *Residual deviance* | *Degrees of freedom* | *Scale factor* |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Level 1A models: Total accidents as a function of traffic flow and link length (1024)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | $\ln k_0$ | 0.7103 | 0.0312 | | | | | | 510.4 | 35 | 17.02 |
| Full | $\ln k_0$ | -10.16 | 1.33 | | | | | | 218.4 | 33 | 7.99 |
| | $\alpha$ | 1.031 | 0.129 | 70.6 | | | | | | | |
| | $\beta$ | 0.623 | 0.0400 | 278.3 | | | | | | | |
| ***Level 1B models: Total accidents as a function of traffic flow, link length and major design variables (1024)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[d15.HARDF\_rhs + a13.Min\_vis]$ | | | | | | | | | | | |
| Full | $\ln k_0$ | -4.19 | 1.49 | | | | | | 114.9 | 31 | 4.626 |
| | $\alpha$ | 0.529 | 0.141 | 14.8 | | | | | | | |
| | $\beta$ | 0.624 | 0.0407 | 275.6 | | | | | | | |
| | HARDF_rhs | -0.577 | 0.0701 | 71.2 | | | Factor | -44% | | | |
| | Min_vis | -0.00185 | 0.00042 | 19.1 | 1.37 | 0.65 | -100m | +20% | | | |
| ***Level 1C models: Total accidents as a function of traffic flow, link length and extra design variables (1024)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[d15.HARDF\_rhs + a13.Min\_vis + d7.DSGNSP]$ | | | | | | | | | | | |
| Full | $\ln k_0$ | -4.56 | 1.49 | | | | | | 98.23 | 30 | 4.212 |
| | $\alpha$ | 0.548 | 0.141 | 15.8 | | | | | | | |
| | $\beta$ | 0.558 | 0.0428 | 193.9 | | | | | | | |
| | HARDF_rhs | -0.664 | 0.0723 | 86.8 | | | Factor | -49% | | | |
| | Min_vis | -0.00172 | 0.000413 | 17.1 | 1.34 | 0.67 | -100m | +19% | | | |
| | DSGSP3 | 0.320 | 0.0787 | 16.7 | | | 120B:120A | +38% | | | |

1.031. However, it should be noted that in spite of the magnitude of the change it was not statistically significant at the 5 per cent level.

The value of the hardstrip parameter suggested that the presence of a hardstrip on the right hand side was associated with an decrease in accidents of 43.9 per cent. The minimum visibility parameter suggested that reducing minimum visibility on the link (i.e. worsening the visibility) by 100m increased accidents by 20.3 per cent.

## 16.3 Level 1C models for D3 links

In Level 1C, fifteen extra variables, comprising more detailed alignment measures and estimates of vehicle speed, as listed in Section 14.6, were added to the final Level 1B model. The results are included in Table 16.1.

Of the variables added in Level 1C, only design speed was significant at the 5 per cent level, and at that the significance was only borderline. It was represented by a 2-level factor DGNSP3, since only two design speeds (120A and 120B) occurred on the 36 D3 links. Design speed and Hardf_rhs were slightly correlated. The best fit model was therefore:

$$\ln(A) = (-4.56 - 0.0183t) + 0.548.\ln(Q) + 0.558.\ln(L_L) + d15.HARDF\_rhs - 0.00172.Min\_vis + d16.DGNSP3,$$

or, in its exponent form,

$$A = 0.0105.e^{-0.0183t}.Q^{0.548}.L_L^{0.558}.\exp(d15.HARDF\_rhs - 0.00172.Min\_vis + d16.DGNSP3),$$

where d15 = 0 for level 1 of factor HARDF_rhs (No offside hardstrip),

d15 = -0.664 for level 2 of factor HARDF_rhs (Offside hardstrip present),

and d16 = 0 for Design speed of 120A,

d16 = 0.320 for Design speed of 120B.

The flow parameter $\alpha$ was not significantly different from 1 but the length parameter $\beta$ was significantly different (at the 0.1 per cent level).

As with Level 1B, the presence of hardstrips on the right hand side was associated with an decrease in accidents, of 48.5 per cent. Reducing the minimum visibility by 100m would increase accidents by 18.8 per cent. Links with design speed 120B, with a higher bendiness value, were associated with 37.7 per cent more accidents than the straighter links with design speed 120A.

## 16.4 Level 2 models for D3 links: Accidents by accident class as function of traffic flow, link length and major design variables

The Level 2 model results for D3 links and accident classes 1 to 3 are shown in Table 16.2.

### *16.4.1 Level 2 models for D3 links: Class 1: Single-vehicle accidents*

There were 323 accidents in the class.

The most significant variable (at the 0.1 per cent level) was the presence of a safety fence. However, its inclusion in the model led to large instabilities and produced a negative flow parameter (which implies that accidents

**Table 16.2 Level 2 models (D3 links): Accidents by accident class as a function of traffic flow, link length and design variables**

| Model | Model terms | Parameter value | s.e. | Deviance difference | *Multiplicative effect compared to mean at:* Min | Max | *% change in accidents for unit change in variable* Unit | %change | Residual deviance | Degrees of freedom | Scale factor |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Class 1: Accidents involving a single vehicle (323)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | $\ln k_0$ | -0.444 | 0.0556 | | | | | | 245.5 | 35 | 8.73 |
| 2A | $\ln k_0$ | -9.48 | 2.29 | | | | | | 85.07 | 33 | 3.278 |
| | $\alpha$ | 0.828 | 0.222 | 14.87 | | | | | | | |
| | $\beta$ | 0.876 | 0.0780 | 160.3 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{(a + b.HARDF\_rhs)}.L_L^{\beta}.\exp[d15.HARDF\_rhs + d16.DGNSP3 + a_{14}.Cminr]$ | | | | | | | | | | | |
| Full | $\ln k_0$ | 5.47 | 3.77 | | | | | | 29.55 | 29 | 1.288 |
| | a | -0.654 | 0.371 | 2.86 | | | | | | | |
| | $\beta$ | 0.882 | 0.0896 | 117.6 | | | | | | | |
| | HARDF_rh | -18.65 | 4.94 | 13.47 | (1) | (1) | | (1) | | | |
| | b | 1.764 | 0.487 | 12.48 | (1) | (1) | | (1) | | | |
| | DGNSP3 | 0.592 | 0.163 | 13.76 | | | 120B:120A | +81% | | | |
| | Cminr | 2.363 | 0.891 | 5.66 | 0.89 | 9.11 | 10 deg/km | +51% | | | |
| ***Class 2: Accidents involving 2 vehicles (400)*** | | | | | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | $\ln k_0$ | -0.230 | 0.0500 | | | | | | 257.4 | 35 | 8.953 |
| 2A | $\ln k_0$ | -14.89 | 2.28 | | | | | | 161.5 | 33 | 5.898 |
| | $\alpha$ | 1.411 | 0.222 | 47.9 | | | | | | | |
| | $\beta$ | 0.515 | 0.0625 | 74.5 | | | | | | | |
| $A = k_0. e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[d15.HARDF\_rhs + a_{13}.Min\_vis + d16.DGNSP3]$ | | | | | | | | | | | |
| Full | $\ln k_0$ | -7.60 | 2.57 | | | | | | 63.11 | 30 | 2.446 |
| | $\alpha$ | 0.802 | 0.245 | 11.86 | | | | | | | |
| | $\beta$ | 0.409 | 0.0667 | 40.8 | | | | | | | |
| | HARDF_rh | -0.915 | 0.119 | 63.4 | | | Factor | -60% | | | |
| | Min_vis | -0.00297 | 0.000646 | 20.93 | 1.658 | 0.506 | -100m | +35% | | | |
| | DGNSP3 | 0.460 | 0.122 | 14.56 | | | Factor | +58% | | | |
| ***Class 3: Accidents involving 3 or more vehicles (186)*** | | | | | | | | | | | |
| $A = k_0. e^{\theta t}.Q^{\alpha}.L_L^{\beta}$ | | | | | | | | | | | |
| Null | $\ln k_0$ | -0.995 | 0.0733 | | | | | | 154.4 | 35 | 4.939 |
| 2A | $\ln k_0$ | -18.76 | 3.52 | | | | | | 99.09 | 33 | 3.156 |
| | $\alpha$ | 1.711 | 0.341 | 30.7 | | | | | | | |
| | $\beta$ | 0.569 | 0.0932 | 41.0 | | | | | | | |
| $A = k_0.e^{\theta t}.Q^{\alpha}.L_L^{\beta}.\exp[a_{10}.Propg\_4 + a_{11}.Max\_vis]$ | | | | | | | | | | | |
| Full | $\ln k_0$ | -8.57 | 3.79 | | | | | | 56.90 | 31 | 1.922 |
| | $\alpha$ | 0.926 | 0.349 | 7.70 | | | | | | | |
| | $\beta$ | 0.641 | 0.112 | 36.99 | | | | | | | |
| | Propg_4 | 2.060 | 0.373 | 26.38 | 0.90 | 7.05 | 10% | +23% | | | |
| | Max_vis | -0.00512 | 0.00164 | 9.63 | 6.12 | 0.79 | -100m | +67% | | | |

*(1) Because of interaction terms the multiplicative effects cannot be shown in a simple form. See text.*

would decrease as traffic flow increased). Since this factor had not appeared in the D3 Level 1 models, its inclusion was rejected in favour of the next most significant variable (at the 1 per cent level), the presence of an offside hardstrip with which the presence of safety fences was correlated. Presence of offside hardstrip (HARDF_rh) also produced a negative flow parameter, but its interaction term with traffic flow was significant at the 1 per cent level, indicating that the effect of a hardstrip varies with traffic flow. For a flow of 30,000 vehicles per day a link with a hardstrip would have 37 per cent fewer accidents than a link with no hardstrip. This figure was reasonable compared to previous models, so the hardstrip factor and its interaction with traffic flow were included in the model. For a flow of 14,000 vehicles per day (close to the mean for all links) a link with a hardstrip would have 84 per cent fewer accidents. The latter figure was very large and, coupled with the fact that the magnitude of the effect was very sensitive to the values of the parameters, indicated that the model was not robust.

Of the other variables Design speed (represented by the factor DGNSP3) and minimum curvature (maximum radius) were found to be significant, the former at the 1 per cent level and the latter at the 5 per cent level. The best fit model was therefore:

$$\ln(A) = (5.47 - 0.0183t) + \alpha.\ln(Q) + 0.882.\ln(L_L) + d15.HARDF\_rhs + d16.Dgnsp3 + 2.363.Cminr,$$

or, in its exponent form,

$$A = 4.211E\text{-}3.e^{-0.0183t}.Q^{\alpha}.L_L^{0.882}.\exp(d15.HARDF\_rhs + d16.Dgnsp3 + 2.363.Cminr),$$

where

| | |
|---|---|
| d15 = 0 and α = - 0.654 | for level 1 of factor HARDF_rhs (No offside hardstrip), |
| d15 = -18.65 and α = 1.110 | for level 2 of factor HARDF_rhs (Offside hardstrip present), |
| and d16 = 0 | for Design speed of 120A, |
| d16 = 0.592 | for Design speed of 120B. |

The flow parameter α was significantly different from 1 (at the 0.1 per cent level), but the length parameter β was not.

The model was not very robust, so care must be taken in interpretation. In particular, there seems no clear reason why the effect of a hardstrip should depend on traffic flow. One possible explanation is that in the limited sample of D3 schemes, only 2 had offside hardstrips, and these had large traffic flows, so there was a correlation which would affect the models. For a flow of 30,000 the results indicated that a link with no hardstrip would have 59.2 per cent more accidents than a link with a hardstrip.

Compared to a link with design speed 120A, a link with design speed 120B (implying a greater bendiness) would have 80.8 per cent more accidents.

An increase in minimum curvature of 10 degrees/km (from straight to 5700m radius, for example) would increase accidents by 51.0 per cent.

### 16.4.2 *Level 2 models for D3 links: Class 2: 2-vehicle accidents*

There were 400 accidents in the class.

The most significant variable (at the 0.1 per cent level) was the presence of an offside hardstrip. As with earlier D3 models, its inclusion reduced the traffic flow parameter, but only from 1.411 to 0.971, and its interaction with traffic flow was not significant. It was therefore included in the model. The other significant variables were minimum visibility (at the 1 per cent level) and Design speed (borderline significant at the 1 per cent level). The latter was represented by the 2-level factor DGNSP3 used in Level 1C. The best fit model was:

$$\ln(A) = (-7.60 - 0.0183t) + 0.802.\ln(Q) + 0.409.\ln(L_L) + d15.HARDF\_rhs - 0.00297.Min\_vis + d16.DGNSP3$$

or, in its exponent form,

$$A = 5.00E\text{-}4.e^{-0.0183t}.Q^{0.802}.L_L^{0.409}.\exp(d15.HARDF\_rhs - 0.00297.Min\_vis + d16.DGNSP3),$$

| | | |
|---|---|---|
| where | d15 = 0 | for level 1 of factor HARDF_rhs (No offside hardstrip), |
| | d15 = -0.915 | for level 2 of factor HARDF_rhs (Offside hardstrip present), |
| and | d16 = 0 | for Design speed of 120A, |
| | d16 = 0.460 | for Design speed of 120B. |

The flow parameter α was not significantly different from 1 but the length parameter βwas significantly different (at the 0.1 per cent level).

The parameters were of the same order as for Level 1C. The absence of a hardstrip would more than double the numbers of accidents. The HARDF_rhs parameter for this model was not significantly different from that in the final Level 1C model. Reducing the minimum visibility by 100m increased accidents by 34.6 per cent. A link with design speed 120B would have 58.4 per cent more accidents than a link with design speed 120A.

### 16.4.3 *Level 2 models for D3 links: Class 3: 3 or more vehicle accidents*

There were 186 accidents in the class.

The most significant variable (at the 0.1 per cent level) was the proportion of link with a steep gradient (steeper than |4| per cent). The only other significant variables (at the 5 per cent level) were maximum visibility and the proportion of offside hardstrip, but the latter was only borderline significant, and introduced instabilities into the model, so was not included. The best fit model was:

$$\ln(A) = (-8.57 - 0.0183t) + 0.926.\ln(Q) + 0.641.\ln(L_L) + 2.060.Propg\_4 - 0.00512.Max\_vis,$$

or, in its exponent form,

$$A = 1.90E\text{-}4.e^{-0.0183t}.Q^{0.926}.L_L^{0.641}.\exp(2.060.Propg\_4 - 0.00512.Max\_vis),$$

The flow parameter α was not significantly different from 1 but the length parameter β was significantly different (at the 5 per cent level).

A link with gradient steeper than 4 per cent (up or down) along one-tenth of its length would have 22.9 per cent more accidents than a link with gradient outside this range. Decreasing the maximum visibility on a link by 100m (i.e. reducing the visibility on the link) increased accidents by 66.9 per cent, which was exceptionally high.

## 17 Summary and discussion

The previous sections 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.

Tables 17.1, 17.2 and 17.3 show summaries of the results for each model, giving the percentage change in the number of accidents for a unit change in each variable. The variables are grouped to bring together all variables relating to edge treatment, curvature and other similar sets.

**Table 17.1 Summary of results from tabulation analysis and Level 1 models for 2-lane dual carriageway**

| | *Symbol* | *Unit* | *Tabulations Link Acc Rate* | *Time trend model* | *1A* | *1B* | *1C* |
|---|---|---|---|---|---|---|---|
| ***Number of accidents*** | | | 2565 | 4688 | 4688 | 4688 | 4688 |
| ***Basic model variables*** | | | | | | | |
| Traffic | α | index | Assumed=1 | 0.808 | 0.808 | 0.804 | 0.804 |
| Link length | β | index | Assumed=1 | | 0.695 | 0.686 | 0.686 |
| ***Major design variables*** | | | | | | | |
| Presence of hardstrip on nearside | HARDF_LHS | Yes:no | -14 | | | | |
| Presence of hardstrip on offside | HARDF_RHS | Yes:no | | | | | |
| Proportion with hardstrip on nearside | HARDP_LHS | tenth | | | | | |
| Proportion with hardstrip on offside | HARDP_RHS | tenth | | | | | |
| Presence of kerb on nearside | KERBF_LHS | Yes:no | +31 | | | | |
| Presence of kerb on offside | KERBF_RHS | Yes:no | | | | | |
| Proportion with kerb on nearside | KERBP_LHS | tenth | | | | | |
| Proportion with kerb on offside | KERBP_RHS | tenth | | | | | |
| Presence of kerbs on nearside or offside | KERBS | Yes:no | | | | | |
| Presence of kerb with no hardstrip | CROSS8 | Yes:no | | | | +31 | +31 |
| Presence of kerb with no continuous white line | KERBL | Yes:no | | | | | |
| Presence of median safety fence | CRASH_F | Yes:no | +10 | | | | |
| Proportion with median safety fence | CRASH_P | tenth | | | | | |
| Presence of continuous edge marking on nearside | CWHTF_LHS | Yes:no | | | | | |
| Presence of continuous edge marking on offside | CWHTF_RHS | Yes:no | | | | | |
| Proportion with continuous edge marking on nearside | CWHTP_LHS | tenth | | | | | |
| Proportion with continuous edge marking on offside | CWHTP_RHS | tenth | | | | | |
| Proportion with continuous obstruction on nearside | OBSTRP_LHS | tenth | | | | | |
| Proportion with continuous obstruction on offside | OBSTRP_RHS | tenth | | | | +15 | +15 |
| Mean verge width on nearside | VW_LHS | m | | | | | |
| Mean verge width on offside | VW_RHS | m | | | | | |
| Bendiness | BENDI | 10 deg/km | | | | +2.9 | +2.9 |
| Hilliness | HILLI | 1 m/km | | | | +0.9 | +0.9 |
| Net gradient | NETGRAD | percent | | | | | |
| Visibility | VISI | -100m | | | | | |
| Minimum visibility | MIN_VIS | -100m | | | | | |
| Maximum visibility | MAX_VIS | -100m | | | | | |
| No of accesses on nearside | ACC_LHS | No/link | | | | | |
| No of accesses on offside | ACC_RHS | No/link | | | | +3.7 | +3.7 |
| Quality of scheme | QUALITY | Good1:good2:lower:urban | | | | | |
| | QUAL4 | Lower+Urban:Good+Best | | | | -25 | -25 |
| | QUAL6 | Lower+Urban+Good:Best | | | | | |
| | QUAL8 | Urban:Good+Best+Lower | | | | | |
| | QUAL9 | Good:Best+Lower+Urban | | | | | |
| | QUAL11 | Good+Lower:Best+Urban | | | | | |
| Quality and presence of hardstrip on offside | CROSSQU2 | Best/Good with HS:Others | | | | | |
| ***Additional design variables*** | | | | | | | |
| COBA-9 speed | COBA-9 | km/h | | | | | |
| COBA-10 speed | COBA-10 | km/h | | | | | |
| V50,wet speed | V50WET | km/h | | | | | |
| Design speed | DGNSPEED | 100A,100B,120A,120B | | | | | |
| | DGNSPD2 | 100:120 | | | | | |
| | DGNSPD3 | 120B:120A | | | | | |
| Maximum curvature | CMAXR | 10 deg/km | | | | | |
| Minimum curvature | CMINR | 10 deg/km | | | | | |
| Proportion with curvature in range C | PROPC_C | tenth | | | | | |
| Proportion with curvature in range D or sharper | PROPC_D | tenth | | | | | |
| Up-hilliness | UPHILLI | 1 m/km | | | | | |
| Down-hilliness | DNHILLI | 1 m/km | | | | | |
| Maximum gradient | GMAXR | % | | | | | |
| Minimum gradient | GMINR | % | | | | | |
| Proportion with gradient >\| 4 \|% | PROPG_4 | tenth | | | | | |
| Proportion with gradient >+ 4% | PROPG_P4 | tenth | | | | | |
| Proportion with gradient <- 4% | PROPG_N4 | tenth | | | | | |

*Shaded areas indicate which variables were tested - if no value is given, the variable was insignificant at the 5 per cent level.*
*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 safety fence, an additional access on a link) or for first category compared with second (eg hardstrip compared with no hardstrip)*

**Table 17.2 Summary of results from Level 2 models for 2-lane dual carriageway**

| | *Symbol* | *Unit* | *1-veh -icle* | *2-veh -icle* | *3+veh -icle* | *J Pedes -trian* | *K Acc -ess* | *L n/s bend* | *M n/s no bend* | *N o/s bend* | *O n/s no bend* | *R other 1-veh* | *S par -ked* | *T over -taking* | *V stop -ped* | *W turn -ing* | *X cross -over* | *Y other bend* | *Z othr> 1-veh* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Number of accidents*** | | | 1731 | 1648 | 590 | 228 | 85 | 133 | 762 | 65 | 425 | 345 | 406 | 741 | 419 | 129 | 238 | 47 | 658 |
| ***Basic model variables*** | | | | | | | | | | | | | | | | | | | |
| Traffic | α | index | 0.428 | 0.839 | 1.229 | 0.902 | 0.945 | 1.000 | 0.553 | 0.812 | 0.525 | 0.315 | 1.022 | 0.866 | 1.305 | 1.000 | 0.630 | 1.000 | 1.114 |
| Link length | β | index | 0.820 | 0.679 | 0.586 | 0.643 | -0.217 | 0.940 | 0.765 | 0.991 | 0.860 | 0.897 | 0.752 | 0.627 | 0.587 | 0.349 | 0.914 | 0.434 | 0.745 |
| ***Major design variables*** | | | | | | | | | | | | | | | | | | | |
| Presence of hardstrip on nearside | HARDF_LHS | Yes:no | | | | -45 | | | | | | | | | | | | | |
| Presence of hardstrip on offside | HARDF_RHS | Yes:no | | | | | | | | | | | | | | | | | |
| Proportion with hardstrip on nearside | HARDP_LHS | tenth | | | | | | | | | | | | | | | | | |
| Proportion with hardstrip on offside | HARDP_RHS | tenth | | | | | | | | | | | | | | | | | |
| Presence of kerb on nearside | KERBF_LHS | Yes:no | | | | | | | | | | | | | | | | | +36 |
| Presence of kerb on offside | KERBF_RHS | Yes:no | | | +64 | | | | | | | | | | | | | | |
| Proportion with kerb on nearside | KERBP_LHS | tenth | | | | | | | | | | | | | | | | | |
| Proportion with kerb on offside | KERBP_RHS | tenth | | | | | | | | | | | | | +5 | | | | |
| Presence of kerbs on nearside or offside | KERBS | Yes:no | | | | | +123 | | | | | | | | | | | | |
| Presence of kerb with no hardstrip | CROSS8 | Yes:no | +26 | +28 | | | | | | | +72 | | | +36 | | | | | |
| Presence of kerb with no continuous white line | KERBL | Yes:no | | | | | | | | | | +62 | | | | | | | |
| Presence of median safety fence | CRASH_F | Yes:no | | | | | -57 | | | | | | | | | | -33 | | |
| Proportion with median safety fence | CRASH_P | tenth | | | | | | | | | | | | | | | | | +2 |
| Presence of continuous edge marking on nearside | CWHTF_LHS | Yes:no | | | | | | | | | | | | | | | | | |
| Presence of continuous edge marking on offside | CWHTF_RHS | Yes:no | | | | | | | | | | | | | | | | | |
| Proportion with continuous edge marking on nearside | CWHTP_LHS | tenth | | | | | | | | | | | | | | | | | |
| Proportion with continuous edge marking on offside | CWHTP_RHS | tenth | | | | | | | | | | | | | | | | | |
| Proportion with continuous obstruction on nearside | OBSTRP_LHS | tenth | | | | | | | | | | | | | | | | | |
| Proportion with continuous obstruction on offside | OBSTRP_RHS | tenth | | +16 | +31 | | | | | | | +29 | | +32 | +37 | | | | |
| Mean verge width on nearside | VW_LHS | m | | | | | | | | | | | | | | | | | |
| Mean verge width on offside | VW_RHS | m | | | | | | | | | | | | | | | | | |
| Bendiness | BENDI | 10 deg/km | +5.0 | +3.8 | | | | +25 | | +21 | | +6.2 | | | | | | | |
| Hilliness | HILLI | 1 m/km | | | +1.4 | | | | | +3.5 | | | | | | | +2 | | |
| Net gradient | NETGRAD | percent | | | | | | | | | | | | | | | | | |
| Visibility | VISI | -100m | | | | | | | | | | | | | | | | | |
| Minimum visibility | MIN_VIS | -100m | | | | | | | | | | | | | | | | | |
| Maximum visibility | MAX_VIS | -100m | | | | +25 | | | | | | | | | | | | | |

| | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| No of accesses on nearside | ACC_LHS | No/link | | | | | +26 | | | | | | | | | | | | |
| No of accesses on offside | ACC_RHS | No/link | | | +7.8 | | | | | | | | +10 | | | +17 | | | |
| Quality of scheme | QUALITY | Good1:good2:lower:urban | | | | | | | | | | | | | | | | | |
| | QUAL4 | Lower+Urban:Good+Best | | -29 | -29 | | | -62 | | | | | -31 | -21 | -30 | | | -53 | |
| | QUAL6 | Lower+Urban+Good:Best | | | | | | | -24 | | -37 | | | | | | | | -28 |
| | QUAL8 | Urban:Good+Best+Lower | | | | | | | | | | +79 | | | | | | | |
| | QUAL9 | Good:Best+Lower+Urban | | | | | +166 | | | | | | | | | | | | |
| | QUAL11 | Good+Lower:Best+Urban | -26 | | | | | | | | | | | | | | | | |
| Quality and presence of hardstrip on offside | CROSSQU2 | Best/Good with HS:Others | | | | | | | | | | | | | | | +92 | | |
| ***Additional design variables*** | | | | | | | | | | | | | | | | | | | |
| COBA-9 speed | COBA-9 | km/h | | | | | | | | | | | | | | | | | |
| COBA-10 speed | COBA-10 | km/h | | | | | | | | | | | | | | | | | |
| V50,wet speed | V50WET | km/h | | | | | | | | | | | | | | | | | |
| Design speed | DGNSPEED | 100A,100B,120A,120B | | | | | | | -62 | | | | | | | | | | |
| | DGNSPD2 | 100:120 | | | | | | | | | | | | | | | | | |
| | DGNSPD3 | 120B:120A | | | | | | | | | | | | | | | | | |
| Maximum curvature | CMAXR | 10 deg/km | | | | | | | | | | | | | | | | | |
| Minimum curvature | CMINR | 10 deg/km | | | | | | | | | | | | | | | | | |
| Proportion with curvature in range C | PROPC_C | tenth | | | | | | | | | | | +6.3 | | | | | | |
| Proportion with curvature in range D or sharper | PROPC_D | tenth | | | | | | | | | | | | | | | | +26 | |
| Up-hilliness | UPHILLI | 1 m/km | | | | | | | | | | | | | | | | | |
| Down-hilliness | DNHILLI | 1 m/km | | | | | | | | | | | +1.7 | | | | | | |
| Maximum gradient | GMAXR | % | | | | | | | | | | | | | | | | | |
| Minimum gradient | GMINR | % | | | | | | | | | | | | | | | | | |
| Proportion with gradient >\| 4 \|% | PROPG_4 | tenth | | | | +11 | | | | | | | | | | | | | |
| Proportion with gradient >+ 4% | PROPG_P4 | tenth | | | | | | | | | | | | | | | | | |
| Proportion with gradient <- 4% | PROPG_N4 | tenth | | | | | | | | | | | | | | | | | |

*Shaded areas indicate which variables were tested - if no value is given, the variable was insignificant at the 5 per cent level.*
*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 safety fence, an additional access on a link) or for first category compared with second (eg hardstrip compared with no hardstrip)*

## Table 17.3 Summary of results from Level 1 and Level 2 models for 3-lane dual carriageway

| | *Symbol* | *Unit* | *1A* | *1B* | *1C* | *1-veh -icle* | *2-veh -icle* | *3+veh -icle* |
|---|---|---|---|---|---|---|---|---|
| ***Number of accidents*** | | | 1024 | 1024 | 1024 | 323 | 400 | 186 |
| ***Basic model variables*** | | | | | | | | |
| Traffic | α | index | 1.031 | 0.529 | 0.548 | -0.654 | 0.802 | 0.926 |
| Link length | β | index | 0.623 | 0.624 | 0.558 | 0.882 | 0.409 | 0.641 |
| ***Major design variables*** | | | | | | | | |
| Presence of hardstrip on nearside | HARDF_LHS | Yes:no | | | | | | |
| Presence of hardstrip on offside | HARDF_RHS | Yes:no | | -44 | -49 | Inter- | -60 | |
| Proportion with hardstrip on nearside | HARDP_LHS | tenth | | | | action | | |
| Proportion with hardstrip on offside | HARDP_RHS | tenth | | | | effect | | |
| Presence of kerb on nearside | KERBF_LHS | Yes:no | | | | | | |
| Presence of kerb on offside | KERBF_RHS | Yes:no | | | | | | |
| Proportion with kerb on nearside | KERBP_LHS | tenth | | | | | | |
| Proportion with kerb on offside | KERBP_RHS | tenth | | | | | | |
| Presence of kerbs on nearside or offside | KERBS | Yes:no | | | | | | |
| Presence of kerb with no hardstrip | CROSS8 | Yes:no | | | | | | |
| Presence of kerb with no continuous white line | KERBL | Yes:no | | | | | | |
| Presence of median safety fence | CRASH_F | Yes:no | | | | | | |
| Proportion with median safety fence | CRASH_P | tenth | | | | | | |
| Presence of continuous edge marking on nearside | CWHTF_LHS | Yes:no | | | | | | |
| Presence of continuous edge marking on offside | CWHTF_RHS | Yes:no | | | | | | |
| Proportion with continuous edge marking on nearside | CWHTP_LHS | tenth | | | | | | |
| Proportion with continuous edge marking on offside | CWHTP_RHS | tenth | | | | | | |
| Proportion with continuous obstruction on nearside | OBSTRP_LHS | tenth | | | | | | |
| Proportion with continuous obstruction on offside | OBSTRP_RHS | tenth | | | | | | |
| Mean verge width on nearside | VW_LHS | m | | | | | | |
| Mean verge width on offside | VW_RHS | m | | | | | | |
| Bendiness | BENDI | 10 deg/km | | | | | | |
| Hilliness | HILLI | 1 m/km | | | | | | |
| Net gradient | NETGRAD | percent | | | | | | |
| Visibility | VISI | -100m | | | | | | |
| Minimum visibility | MIN_VIS | -100m | | +20 | +19 | | +35 | |
| Maximum visibility | MAX_VIS | -100m | | | | | | +67 |
| No of accesses on nearside | ACC_LHS | No/link | | | | | | |
| No of accesses on offside | ACC_RHS | No/link | | | | | | |
| Quality of scheme | QUALITY | Good1:good2:lower:urban | | | | | | |
| | QUAL4 | Lower+Urban:Good+Best | | | | | | |
| | QUAL6 | Lower+Urban+Good:Best | | | | | | |
| | QUAL8 | Urban:Good+Best+Lower | | | | | | |
| | QUAL9 | Good:Best+Lower+Urban | | | | | | |
| | QUAL11 | Good+Lower:Best+Urban | | | | | | |
| Quality and presence of hardstrip on offside | CROSSQU2 | Best/Good with HS:Others | | | | | | |
| ***Additional design variables*** | | | | | | | | |
| COBA-9 speed | COBA-9 | km/h | | | | | | |
| COBA-10 speed | COBA-10 | km/h | | | | | | |
| V50,wet speed | V50WET | km/h | | | | | | |
| Design speed | DGNSPEED | 100A,100B,120A,120B | | | | | | |
| | DGNSPD2 | 100:120 | | | | | | |
| | DGNSPD3 | 120B:120A | | | +38 | +81 | +58 | |
| Maximum curvature | CMAXR | 10 deg/km | | | | | | |
| Minimum curvature | CMINR | 10 deg/km | | | | +51 | | |
| Proportion with curvature in range C | PROPC_C | tenth | | | | | | |
| Proportion with curvature in range D or sharper | PROPC_D | tenth | | | | | | |
| Up-hilliness | UPHILLI | 1 m/km | | | | | | |
| Down-hilliness | DNHILLI | 1 m/km | | | | | | |
| Maximum gradient | GMAXR | % | | | | | | |
| Minimum gradient | GMINR | % | | | | | | |
| Proportion with gradient >\| 4 \|% | PROPG_4 | tenth | | | | | | +23 |
| Proportion with gradient >+ 4% | PROPG_P4 | tenth | | | | | | |
| Proportion with gradient <- 4% | PROPG_N4 | tenth | | | | | | |

*Shaded areas indicate which variables were tested - if no value is given, the variable was insignificant at the 5 per cent level.*
*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 safety fence, an additional access on a link) or for first category compared with second (eg hardstrip compared with no hardstrip)*

Table 17.1 covers the accident rate analysis, time trend models and Level 1 models for D2 links. Table 17.2 covers the Level 2 models for D2 links. Table 17.3 covers the Level 1 and 2 models for D3 links.

### 17.1 Traffic flow

The value of the traffic parameter $\alpha$ was less than 1 in almost all models (except for some non-robust models where it was set to 1). This implies that accident frequency would increase as traffic increased, but less than in direct proportion. Put another way, accident *rate* would decrease as traffic flow increased. This effect has been found in many similar studies.

The parameter in the single-vehicle accident groups was lower than in the multi-vehicle accident groups. Its value was 0.43 for all single-vehicle accidents together, 0.84 for 2-vehicle accidents, and 1.23 for accidents involving 3 or more vehicles. This is reasonable, as the heavier the traffic, the more likely is it that a vehicle losing control would hit one or more other vehicles.

### 17.2 Link length

The parameter for link length $\beta$ was consistently less than 1, implying that longer links had a lower accident density than shorter ones. This is attributable to a spill-over effect from the junctions at the ends of the links. There was no significant difference in the effect of link length on single- and multi-vehicle accidents, both being about 0.7.

### 17.3 Time effects

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.

#### *17.3.1 Variation of accident risk with time*

The variation over time when there was no change of traffic flow or layout was found to show a decrease of about 2 per cent per year (with a standard error of 0.4 per cent). This could be attributable to improvements in vehicle design or driver behaviour.

This result appears at odds with the findings from the single-carriageway study, where the change with time was found to be an increase of 1.7 per cent per year. However, when sampling errors, and the larger proportion of older schemes in the dual-carriageway study, were taken into account, the results were not dissimilar.

There was slight evidence that the decrease was greater on 2-lane and less on 3-lane dual carriageways.

#### *17.3.2 Variation of accident risk over time with traffic growth*

Traffic growth across all schemes averaged 4.5 per cent per year between 1979 and 1990. However, because the relationship between traffic flow and accident numbers was non-linear, traffic growth produced an increase in accidents of only 3.7 per cent per year.

The variation of accident numbers with traffic flow is often allowed for by analysing accident rates (accidents per million vehicle-km) instead of total numbers. The non-linearity resulted in a decrease in accident rates of about 0.8 per cent per year due to traffic growth.

#### *17.3.3 Variation of accident risk over time with layout*

The effect of road layout on accidents was mainly determined by comparing links with a given feature with those without, that is, by a cross-sectional analysis. Layout also had a time dimension, because as design standards improve, newer roads should, other things being equal, have fewer accidents. This effect could be studied by using the opening year of the scheme as a summary variable for a range of design features.

In this study, scheme opening year had a significant effect on accident rates, with the rate on schemes opened in 1986 or later being 19 per cent lower than on schemes opened between 1981 and 1985. Over 5 years, the time trend would account for a reduction in accident rate of about 10 per cent, and the non-linearity of accidents with traffic growth another 4 per cent, leaving about 5 per cent to be accounted for by changes in layout.

Scheme opening year did not appear as a significant variable in any model where design variables were also included, implying that, as expected, it was the design features which affected accidents.

Thus, from this simple analysis, improvements in road design accounted for a reduction in accidents of about 1 per cent per year. The following sections indicate the features of road design which contributed to this trend.

### 17.4 Road quality

Road quality, which was estimated semi-subjectively from consideration of the numbers and types of junctions, appeared as a significant effect in many models, though the details varied. In several models, which can be described collectively as those involving manoeuvres or avoiding action, roads of Best or Good quality were found to have a higher accident frequency than roads of Lower or Urban quality. In several other models, which can be described as categories where one vehicle lost control, roads of Best quality were found to have a higher frequency than roads of Good, Lower or Urban quality. Although the dividing line was slightly different, these two sets were consistent, and implied that the better quality roads had more accidents (averaging around 25 per cent). This result is thought to be due to higher speeds on better quality roads.

However, the results on road quality were not all consistent. In one group, representing an ‘other’ category for single vehicles, the Urban quality was found to give a higher accident frequency, while for access accidents, the Good category had a higher frequency (though this latter result was not robust).

The above analysis applied only to link accidents. Results from the tabulation analysis (section 12.4) would suggest that when junction accidents were included, the overall accident risk fell as the quality improved. The conclusion is that roads of better design (generally, more modern roads) had fewer accidents because their junctions were fewer and of better design. This conclusion is tentative because it was not possible, without re-formulating the models, to include junction accidents in the regression analysis.

Road quality is therefore an interesting variable which gave some marked, but not completely consistent, results. Further investigation could be useful.

## 17.5 Edge treatment

Edge treatment is a complex factor which was described by a number of different aspects - presence of kerbs, presence of hardstrips and presence of continuous edge lines, each of which could apply to either side of the carriageway and was measured in this study by either a 2-level factor (present/absent) or by a variable denoting the proportion of the link with that feature. These variables and factors were highly correlated with each other, and in different models one or other appeared as most significant, so interpretation was not easy. Nevertheless, a clear message emerged.

In several models, a factor combining the effects of hardstrips and kerbs was found to be significant, and generally implied that a carriageway edge which was level or with kerbs which were not set back resulted in a higher accident rate than an edge where there was a hardstrip alone or a kerb set back behind a hardstrip. For all accidents together links with kerbs which were not set back behind a hardstrip had about 30 per cent more accidents than those without kerbs or with set-back kerbs. When there was a kerb on one side only, it was most often on the offside, so it is not surprising to find that the effect of a kerb was greater in those accident groups where the offside edge would be important - overtaking (kerbs which are not set back give 36 per cent more accidents) and leaving the carriageway on the offside (72 per cent more). Accidents which involved a vehicle stopping or held up were also affected by offside kerbs (increase of 58 per cent), where it is reasonable to suppose that the presence of a kerb affects any avoiding action.

Edge treatment on the nearside of the carriageway affected two general categories (single- and multi-vehicle) of accidents, where the presence of a kerb increased accidents. It also appeared to affect pedestrian accidents (where the presence of a hardstrip reduced accidents), but this was probably due to the fact that where there was a significant flow of pedestrians, a footway and kerb, rather than a hardstrip, was likely to be provided.

Across the majority of the models, therefore, the presence of a kerb was generally associated with an increase in accidents, except when the kerb was set back - behind a hardstrip in some models, or behind a continuous edge line in others. The presence of a hardstrip was generally associated with a reduction in accidents.

## 17.6 Central reservation treatment

### *17.6.1 Obstructions*

The proportion of link with a continuous obstruction on the median was a significant variable in several models. In the model for all accidents together, a 10 per cent increase in this proportion resulted in a 15 per cent increase in accidents. The effect was greater in some individual groups, particularly in those concerned with overtaking or vehicles stopping or held up on the carriageway - another instance where avoiding action was affected by the presence of obstacles.

### *17.6.2 Safety fences*

The presence of a safety fence on the nearside of the carriageway had no significant effect on accidents in the models. This was largely because, apart from localised fencing to protect particular obstructions (bridge piers etc), there were very few links with a substantial proportion of nearside safety fence.

Safety fences on the median were associated with a decrease in accidents where a vehicle crossed the central reserve (a 33 per cent reduction where more than half the link had a fence), and in accidents at accesses (a 57 per cent reduction), but an increase in accidents allocated to the ‘other multi-vehicle accident’ category (an 11 per cent increase where half the link had a fence). This could be because a safety fence can deflect a vehicle back onto the carriageway. On balance, a link with a median safety fence along most of its length would be expected to have about 3 per cent fewer accidents than one without. This small reduction was consistent with the finding that the presence of a safety fence was not a significant factor in the model for all accidents together.

It is important to note, however, that median safety fences are normally located in places where the consequences of a vehicle crossing the central reserve would otherwise be particularly severe, due to the presence of hazards and obstructions. The accident risk at such sites, in the absence of a safety fence, would therefore be far greater than that of the average site where there is no safety fence, so the models do not compare like with like. The presence of a fence would therefore reduce the risk more than the figures would suggest. In addition, when an accident involving a vehicle running off the road onto the median did occur, the severity of a collision with a safety fence would be less than if the vehicle had hit an obstacle or crossed the central reservation.

## 17.7 Alignment

### *17.7.1 Bendiness*

Bendiness was a significant factor affecting accidents which involved a single vehicle leaving the carriageway on a bend (where an increase of 10 degrees/km in bendiness increases accidents by about 20 per cent), and to a lesser extent in single-vehicle accidents where the vehicle did not leave the carriageway (about 6 per cent). Bendiness also appeared in the models for all single-vehicle accidents combined and all 2-vehicle accidents combined, where an increase of 10 degrees/km resulted in an increase in accidents of 5 per cent and 4 per cent respectively.

Thus it appears that bendiness was associated with an increase in accidents of certain types - mainly single-vehicle accidents and particularly those that involve a vehicle leaving the road on a bend - but not in accidents of other types.

#### *17.7.2 Hilliness*

Hilliness was a significant factor affecting accidents in two similar groups - that which involved a single vehicle leaving the carriageway offside on a bend, and that which involved a vehicle crossing the central reserve. The parameter value was similar in each case, and implied an increase in accidents of about 2 to 3 per cent for an increase of 1 metre/km in hilliness. Hilliness also appeared in the models for all accidents combined and all for accidents involving 3 or more vehicles, where an increase of 1 metre/km resulted in an increase in accidents of about 1 per cent.

Down-hilliness, which is closely related to hilliness, was associated with a small increase in accidents where a parked vehicle was involved (2 per cent increase for a 1 metre/km increase in down-hilliness). Pedestrian accidents were affected by steep gradients (a 10 per cent increase in the proportion of gradient steeper than |4| per cent resulted in an 11 per cent increase in pedestrian accidents).

#### *17.7.3 Visibility*

The only type of accident where visibility appeared as a significant variable in a consistent way was that involving pedestrians, where a 100m decrease in maximum visibility resulted in a 25 per cent increase in accidents.

### 17.8 Carriageway width

Links with 3 lanes were modelled separately from 2-lane links, and since there were only 36 such links there were few significant variables in the models produced. Such variables as were found to be significant were broadly consistent with the results from the 2-lane links. In particular, the presence of a hardstrip was found to reduce accidents in the 3-lane models, and this was similar to the effects of edge treatment described above. Design speed and minimum visibility were also found to be just significant; these variables were correlated with bendiness which appears in the 2-lane models.

The average accident rate on 3-lane links was similar to that on 2-lane links. However, this average concealed differences between links with hardstrips, where 3-lane links were significantly safer than 2-lane links, and links with kerbs, where 3-lane links were less safe.

The variation of accident frequency with traffic flow was significantly different on 2-lane and 3-lane links.

### 17.9 Accesses

Not surprisingly, accidents involving a turning vehicle and accidents at an access were affected by the number of accesses on a link, with an extra access resulting in about 20 per cent more accidents in these groups. Over all types of accident, an extra access resulted in about 4 per cent more accidents. Accidents involving parked vehicles were also affected.

### 17.10 Speeds

Several variants of a speed variable were used in the models, but in only one model was speed found to be significant. This was the group relating to accidents where a single vehicle left the carriageway on the nearside other than on a bend, where a design speed of 120 km/h was found to be safer than 100 km/h. This result is believed to be due to the strong effect of bendiness in the COBA design speed formula, rather than any effect of speed itself.

## 18 Conclusions

This section summarises the results from a substantive study of accidents on non-built-up dual-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 dual-carriageway trunk roads of modern design.

### 18.1 The study

The study covered 112 schemes, comprising about half of the dual-carriageway trunk road schemes in England which had opened since 1968. There were 9097 personal injury accidents on the schemes studied, of which 5712 were link accidents, 1278 occurred at major junctions and 2107 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.

### 18.2 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 single-carriageway roads (Walmsley, Summersgill and Binch, 1998).

1 The average accident rate for link accidents on the schemes studied was 10.4 accidents per 100 MVkm. Rates for old rural A-roads of traditional design (pre-1968) were around twice as high (22 per 100 MVkm). The conclusion is that modern dual-carriageway trunk roads of the kind studied in this report were safer on

average than traditional A-roads. This result is consistent with the findings from the single-carriageway study.

2 Good, high-speed modern dual carriageways, with few roundabouts and with most junctions grade-separated or having slip roads, had around 25 per cent more *link* accidents than roads with more roundabouts, T-junctions and crossroads, or than modern roads in semi-urban areas, at the same flow level. This is thought to be due to higher speeds on the better-quality roads. However, when junction accidents were taken into account, the overall accident rate on the *scheme* was significantly lower (by about 40 per cent). Thus, good modern roads had fewer accidents because their junctions were fewer and of better design.

3 Roads with kerbs had a higher accident frequency than those without, except where the kerb was set back behind a hardstrip (or in some cases a continuous edge line). The increase in accidents on links with kerbs which were not set back (either on the nearside or the median side) was around 30 per cent in total, and the increase was greater in accident groups which involved a vehicle overtaking, crossing the central reservation or stopping on the carriageway. Put another way, roads with hardstrips or continuous edge lining had about 23 per cent fewer accidents than those with kerbs but without hardstrips, other things being equal.

In the single-carriageway study, it was the presence of a hardstrip rather than the absence of kerbs 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.

4 Dual-carriageway roads with a median safety fence had 33 per cent fewer accidents of the type which involved a vehicle crossing the central reservation, and 57 per cent fewer access accidents, but around 11 per cent more of some other types of accident. Over all accident types, a road with a median safety fence along most of its length would be expected to have about 3 per cent fewer accidents than one without, other things being equal.

However, median safety fences are normally located in places where the consequences of a vehicle crossing the central reserve would otherwise be particularly severe, due to the presence of hazards and obstructions, so the models do not compare like with like. The presence of a fence would reduce the risk more than the figures would suggest. In addition, the severity of a collision with a safety fence would be less than if the vehicle had hit an obstacle or crossed the central reservation.

5 Dual-carriageway roads with a higher proportion of obstructions on the median had a higher accident frequency. For each increase of 10 per cent in the proportion of the link with an obstruction, the accident frequency increased by 15 per cent overall, with greater increases for accident groups involving a vehicle overtaking or stopping on the carriageway.

6 Horizontal alignment had a small effect on accidents. A link with twice the average bendiness would have around 10 per cent more accidents in total, but up to 60 per cent more accidents which involved a vehicle leaving the carriageway. Vertical alignment also had a small effect. A link with twice the average hilliness would have around 14 per cent more accidents in total, but up to 30 per cent more accidents which involved a vehicle leaving the carriageway on a bend or crossing the central reserve.

In the single-carriageway study, horizontal alignment had no significant effect on accidents, and vertical alignment only a very small effect. 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.

7 Any conclusions on the effect of carriageway width can only be tentative, because the sample of wide (3-lane) dual carriageways contained few schemes and was unduly influenced by one scheme with an extremely high traffic flow. From the analysis that was possible, it appeared that average accident frequencies on 3-lane dual carriageways were similar to those on 2-lane schemes, and were affected by mostly the same variables. However, this similarity in average frequencies masked significant differences between links with hardstrips, where 3-lane schemes were significantly safer than those with 2 lanes. The variation of accident frequency with traffic flow was significantly different on 2-lane and 3-lane links.

In the single-carriageway study, wide (WS2) roads were found to have substantially fewer accidents than standard-width (S2) roads at similar traffic levels.

8 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 decrease of about 2 per cent per year; there was slight evidence that the decrease was greater on 2-lane and less on 3-lane dual carriageways. The trend in accident *rates* over time when there was no change of flow or of layout with time was the same -2 per cent per year.

In the single-carriageway study, the underlying trend was found to be an increase of 1.7 per cent per year. However, when sampling errors, and the larger proportion of older schemes in the dual-carriageway study, were taken into account, the results were not dissimilar.

9 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 hardstrips, safety fences and other road design features which contribute to fewer accidents, and to fewer and better-designed junctions. This result is consistent with the findings from the single-carriageway study.

10 The relation between link accident frequency and traffic flow was not linear. Traffic growth across all schemes averaged 4.8 per cent per year between 1979 and 1990, but because of this non-linearity, accidents increased by only 3.9 per cent per year. The non-linearity of accident *numbers* with traffic flow resulted in a decrease in accident *rates* of 0.9 per cent per year due to traffic growth. There was evidence that the non-linearity was greater for accidents involving a single vehicle, less for accidents involving two vehicles, and indistinguishable from linearity for accidents involving three or more vehicles. These results are consistent with the findings from the single-carriageway study.

11 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 7 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 single-carriageway study.

There was evidence that the non-linearity was less for accidents involving a single vehicle. This result was contrary to that found in the single-carriageway study, where the non-linearity was greater for single-vehicle accidents.

12 None of the other variables tested had a clear effect on accident risk. The reason why only a small number of variables appeared to be significant is probably that this study was limited to dual-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.

### 18.3 In conclusion

The foregoing paragraphs summarise the features of dual-carriageway roads which affect accident risk. Apart from these features, the results from the study imply 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 no kerbs, and provided the number of junctions is minimised. 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.

## 19 Acknowledgements

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## Appendix 1: Dual-carriageway schemes in the study

| *Scheme number* | *Road* | *Location* | *County* | *Length (km)* | *Date opened* |
|---|---|---|---|---|---|
| ***Dual-carriageway schemes with 2 lanes (D2) throughout*** | | | | | |
| 1 | A5 | Milton Keynes Diversion | Bucks/Northants | 14.2 | Oct 1980 |
| 6 | A12 | Colchester Northern Bypass | Essex | 7.0 | Jan 1974 |
| 8 | A120 | Colchester Eastern Bypass | Essex | 11.4 | Jun 1982 |
| 9 | A10 | Hoddesdon Bypass | Herts | 6.9 | Jan 1974 |
| 10 | A10 | Ware Bypass | Herts | 7.2 | Aug 1976 |
| 11 | A10 | Between Bullsmoor Lane, Enfield and Turnford | Herts | 4.0 | Jan 1968 |
| 12 | A47 | King’s Lynn Bypass | Norfolk | 3.0 | Jan 1975 |
| 13 | A47 | East Dereham Bypass | Norfolk | 3.4 | Mar 1978 |
| 14 | A11 | Barton Mills Bypass | Suffolk | 4.0 | May 1986 |
| 15 | A12 | Ufford & Wickham Market Bypass | Suffolk | 6.5 | Jul 1976 |
| 16 | A12 | Ipswich Bypass (Eastern Section) | Suffolk | 5.5 | Jun 1984 |
| 17 | A12 | Martlesham Bypass | Suffolk | 2.6 | Jun 1984 |
| 18 | A45 | Kentford to Bury St Edmonds Improvement | Suffolk | 9.8 | Jul 1977 |
| 19 | A45 | Rougham to Woolpit | Suffolk | 7.1 | May 1979 |
| 20 | A45 | Ipswich Bypass (South-East Section) | Suffolk | 6.2 | Sep 1982 |
| 21 | A38 | Derby Ring Road | Derbyshire | 2.4 | Sep 1983 |
| 23 | A38 | Allestree Link | Derbyshire | 1.6 | Jan 1968 |
| 24 | A52 | Borrowash Bypass Extension | Derbyshire | 1.1 | May 1980 |
| 26 | A38 | Little Eaton to Holbrook Improvement | Derbyshire | 4.4 | Oct 1977 |
| 27 | A61 | Ripley-Swanwick-Alfreton Bypass | Derbyshire | 8.1 | Oct 1977 |
| 28 | A61 | Chesterfield Inner Relief Rd, Section 1 | Derbyshire | 6.2 | Jul 1985 |
| 29 | A61 | South Normanton Bypass | Derbyshire | 2.3 | Jan 1968 |
| 30 | A61 | Unstone and Dronfield Bypass | Derbyshire | 6.3 | Jan 1975 |
| 32 | A46 | Syston Western Bypass | Leics | 4.5 | Jan 1969 |
| 33 | A1 | Long Bennington Bypass | Lincolnshire | 3.5 | Jan 1968 |
| 34 | A46 | Lincoln Relief Road | Lincolnshire | 5.5 | Dec 1985 |
| 35 | A43 | Towcester Bypass | Northants | 4.9 | Feb 1988 |
| 36 | A1 | Shire Bridge to Balderton | Notts | 2.7 | Jan 1971 |
| 37 | A614 | Nottingham Ring Road Extension | Notts | 3.9 | Jan 1968 |
| 39 | A19 | Easington and Cold Hesledon Diversions | Durham | 4.8 | Jan 1971 |
| 40 | A66 | Elton Bypass | Durham | 2.5 | Jan 1969 |
| 42 | A67 | Long Newton Bypass | Durham | 1.9 | Jan 1968 |
| 43 | A67 | Sadberge Bypass | Durham | 1.6 | Jan 1968 |
| 44 | A1 | Seaton Burn to Stannington Bridge | Northmblnd | 4.1 | Jan 1969 |
| 46 | A69 | Hexham and Corbridge Bypass | Northmblnd | 12.6 | Feb 1977 |
| 53 | A74 | Greymoorhill to Moss Band | Cumbria | 7.7 | Jan 1971 |
| 54 | A590 | Lindale Bypass | Cumbria | 3.5 | Feb 1977 |
| 58 | A66 | Keswick Northern Bypass | Cumbria | .9 | Jan 1975 |
| 59 | A66 | Palliard Bridge to County Boundary | Cumbria | 3.3 | Dec 1979 |
| 60 | A66 | Appleby Bypass | Cumbria | 6.2 | Apr 1981 |
| 61 | A56 | Edenfield to Rawtenstall Level Crossing Bypass | Lancashire | 2.2 | Jan 1969 |
| 64 | A34 | Sandleford Link | Berkshire | 1.9 | Jul 1979 |
| 65 | A34 | M4 to Ashridge Farm | Berkshire | 6.0 | Sep 1979 |
| 66 | A34 | East Ilsley to Chilton Improvement | Berkshire | 4.2 | Dec 1986 |
| 67 | A23 | Bolney Diversion and Flyover | East Sussex | 1.6 | Jan 1972 |
| 68 | A27 | Lewes Southern Bypass | East Sussex | 3.6 | Dec 1977 |
| 69 | A27 | Falmer Diversion | East Sussex | 5.2 | Dec 1979 |
| 71 | A27 | Havant Bypass to Chichester Bypass | Hants/W. Sussex | 12.4 | Aug 1988 |
| 73 | A303 | Andover to Thruxton | Hampshire | 5.9 | Nov 1985 |
| 74 | A303 | Andover Bypass | Hampshire | 5.8 | Jan 1969 |
| 76 | A34 | Whitchurch and Litchfield Bypass | Hampshire | 8.2 | Jan 1975 |
| 77 | A34 | Kingsworthy-Bullington(Sutton Scotney Bypass) | Hampshire | 9.0 | Oct 1981 |
| 81 | A2 | Bridge Bypass | Kent | 2.9 | Jun 1976 |
| 82 | A2 | Canterbury Bypass | Kent | 8.6 | Oct 1981 |
| 84 | A34 | Oxford Southern Byp,Harcourt Hill to South Hinksey | Oxfordshire | 4.7 | Jan 1974 |
| 91 | A30 | Camborne and Scorrier Bypass | Cornwall | 11.1 | Jan 1975 |
| 93 | A38 | Liskeard Bypass | Cornwall | 2.7 | Jan 1975 |
| 94 | A38 | Trethawle Improvement | Cornwall | 3.2 | Oct 1978 |
| 95 | A38 | Saltash Bypass | Cornwall | 2.5 | Sep 1988 |
| 96 | A30 | Pearces Hill to Pocombe Link | Devon | 5.6 | May 1977 |
| 97 | A30 | Exeter to Okehampton Stage 1 | Devon | 11.9 | May 1978 |
| 98 | A30 | Exeter to Okehampton Stage 2 | Devon | 8.8 | Jun 1978 |
| 99 | A30 | Exeter to Okehampton Stage 3 | Devon | 8.4 | Jul 1987 |
| 100 | A361 | North Devon Link (M5 to Tiverton) | Devon | 10.3 | Mar 1984 |
| 101 | A38 | Chudleigh Bypass | Devon | 4.3 | Jan 1973 |
| 102 | A38 | Ivybridge Bypass | Devon | 7.0 | Jan 1974 |

| Scheme number | Road | Location | County | Length (km) | Date opened |
|---|---|---|---|---|---|
| 103 | A38 | Lee Mill to Smithaleigh | Devon | 2.6 | Jan 1974 |
| 104 | A38 | Marsh Mills to Manadon | Devon | 4.9 | Apr 1985 |
| 105 | A4 | Keynsham Bypass | Somerset | 3.2 | Jan 1968 |
| 106 | A303 | Ilchester Bypass | Somerset | 7.3 | Mar 1977 |
| 107 | A303 | Sparkford Bypass | Somerset | 5.2 | Oct 1989 |
| 109 | A303 | Mere Bypass | Wiltshire | 3.0 | Jul 1976 |
| 110 | A303 | Amesbury Bypass | Wiltshire | 3.2 | Jan 1969 |
| 111 | A303 | Wylye Bypass | Wiltshire | 7.4 | Jan 1975 |
| 112 | A36 | Whaddon to Petersfinger(Alderbury Bypass) | Wiltshire | 4.9 | Sep 1978 |
| 113 | A36 | Beckington Bypass | Wiltshire | 1.7 | Nov 1989 |
| 114 | A419 | Stratton St Margaret Bypass | Wiltshire | 4.5 | Oct 1977 |
| 115 | A419 | Common Head & Covingham Farm Diversion | Wiltshire | 5.0 | Jan 1971 |
| 117 | A449 | Ombersley Bypass | Hereford/Worc | 2.2 | Jan 1974 |
| 118 | A40 | Ross to Monmouth County Boundary | Hereford | 12.5 | Jan 1969 |
| 121 | A5 | Muckley Corner to Wall Bypass | Staffs | 3.6 | Jan 1971 |
| 122 | A38 | Sutton Coldfield Bypass, Canwell Bassetts Pole Divn | Staffs | 6.4 | Jan 1973 |
| 123 | A180 | Brigg to Ulceby | Humberside | 9.3 | Mar 1983 |
| 124 | A180 | Ulceby to Grimsby | Humberside | 12.4 | Dec 1983 |
| 125 | A1 | Scotch Corner Diversion | N Yorks | 1.1 | Jan 1971 |
| 126 | A19 | Cleveland Tontine to Clack Lane End | N Yorks | 2.3 | Jan 1971 |
| 127 | A19 | South of Clack Lane End to North end of Barrowby Divn | N Yorks | 8.7 | Jan 1971 |
| 128 | A64 | York Bypass | N Yorks | 12.9 | Apr 1976 |
| 129 | A64 | Tadcaster Bypass | N Yorks | 6.2 | Sep 1978 |
| 131 | A629 | Airedale Section 1, Kildwick to Beechcliffe | W Yorks | 4.0 | Aug 1988 |
| 132 | A650 | Airedale Section 2, Victoria Park-Crossflatts | W Yorks | 3.1 | Oct 1988 |
| 134 | A11 | Cringleford Bypass | Norfolk | 1.2 | Jan 1975 |
| 135 | A45 | Woolpit to Haughley New St Improvement | Suffolk | 4.9 | Jun 1977 |
| 136 | A1 | Stamford Bypass to The Fox at South Witham | Leics | 15.5 | Jan 1971 |
| 137 | A19 | Wolviston to South of Sheraton Cross Roads, Stage 2 | Durham | 8.8 | Jan 1971 |
| 138 | A590 | Levens Bridge Diversion | Cumbria | 2.2 | Nov 1983 |
| 139 | A56 | Rawtenstall Inner Relief Road | Lancashire | 1.1 | Jan 1969 |
| 140 | A56 | Accrington Eastern Bypass, South Section | Lancashire | 5.2 | Jul 1985 |
| 141 | A38 | Lee Mill to Westover, Ivybridge | Devon | 2.1 | Jan 1968 |
| 142 | A19 | Thirsk Bypass | N Yorks | 4.1 | Jan 1972 |
| 491 | A69 | Throckley, Heddon and Horseley Diversion | Northmblnd | 3.5 | Jan 1975 |
| 561 | A59 | Meathop to Sampool Bridge | Cumbria | 2.0 | Mar 1982 |
| 562 | A590 | Meathop to Sampool Bridge | Cumbria | 1.1 | Mar 1982 |
| ***Dual-carriageway schemes with 3 lanes (D3) throughout*** | | | | | |
| 3 | A45 | Newmarket Bypass | Cambs | 12.9 | Jan 1975 |
| 7 | A12 | Stanway Bypass | Essex | 5.1 | Jan 1971 |
| 80 | A2 | Woolwich boundary to Medway Motor Road, Stage 2 | Kent | 9.6 | Jan 1968 |
| 85 | A3 | Esher Bypass | Surrey | 11.5 | Dec 1976 |
| 87 | A3 | Ripley Bypass | Surrey | 5.7 | Jan 1975 |
| ***Dual-carriageway schemes with some 2-lane (D2) and some 3-lane (D3) links*** | | | | | |
| 22 | A38 | Mickleover Bypass and Link Road | Derbyshire | 5.6 | Jan 1974 |
| 45 | A1 | Morpeth Bypass | Northmblnd | 7.4 | Jan 1971 |
| 86 | A3 | Burpham to Ladymead Diversion | Surrey | 8.0 | Jul 1981 |
| 130 | A1 | Wetherby Bypass | W Yorks | 4.0 | Jul 1988 |

# Appendix 2: Numbers of accidents, by group

| Group | No of vehicles | Description | No of accidents |
|---|---|---|---|
| ***Junction accidents:*** | | | |
| A | All | Pedestrian accidents at junctions | 113 |
| B | 1 | Accident at junction | 688 |
| C | 1 | Accident on internal link | 59 |
| D | 2 or more | Accident at junction | 2414 |
| E | 2 or more | Accident on internal link | 94 |
| All junction accidents | | | 3368 |
| ***Link accidents:*** | | | |
| J | All | Pedestrian accidents on links | 263 |
| K | All | Accident at access road | 90 |
| L | 1 | Left carriageway nearside on a bend | 139 |
| M | 1 | Left carriageway nearside, not on a bend | 909 |
| N | 1 | Left carriageway offside on a bend | 72 |
| O | 1 | Left carriageway offside, not on a bend | 495 |
| R | 1 | Other single-vehicle accident | 437 |
| S | 2 or more | Accident involving parked vehicle | 481 |
| T | 2 or more | One vehicle overtaking or changing lane | 943 |
| V | 2 or more | One vehicle stopping, starting or held up | 548 |
| W | 2 or more | One vehicle turning or waiting to turn | 135 |
| X | 2 or more | One vehicle crossed central reservation | 249 |
| Y | 2 or more | Other accident on bend | 52 |
| Z | 2 or more | Other multi-vehicle accident | 891 |
| All link accidents | | | 5704 |
| Accidents not assigned to a group | | | 25 |
| All accidents | | | 9097 |

# Appendix 3: Numbers of accidents, by class

| Class | No of vehicles | Description | No of accidents |
|---|---|---|---|
| ***Junction accidents:*** | | | |
| 1 | 1 | One-vehicle accidents | 752 |
| 2 | 2 | Two-vehicle accidents | 2185 |
| 3 | 3 or more | Multiple vehicle accidents | 335 |
| All junction accidents | | | 3272 |
| ***Link accidents:*** | | | |
| 1 | 1 | One-vehicle accidents | 2054 |
| 2 | 2 | Two-vehicle accidents | 2048 |
| 3 | 3 or more | Multiple vehicle accidents | 776 |
| All link accidents | | | 4878 |
| Accidents not assigned to a class (including pedestrian, access and stationary vehicle accidents) | | | 947 |
| All accidents | | | 9097 |

# Appendix 4: List of variables measured

## Alignment data

| Variable | Mini -mum | Maxi -mum | Mean |
|---|---|---|---|
| Length (km) | 0.04 | 12.11 | 1.91 |
| Bendiness (°/km) | 0 | 195 | 35.1 |
| Maximum curvature (°/km) | 0 | 1900 | 128 |
| equivalent to minimum radius of: | straight | 30m | 450m |
| Proportion in range C[1] | 0% | 100% | 32% |
| Propn in range D or sharper[2] | 0% | 100% | 31% |
| Hilliness (m/km) | 0 | 50.7 | 14.1 |
| Up-hilliness (m/km) | 0 | 50.7 | 7.1 |
| Down-hilliness (m/km) | 0 | 48.6 | 7.0 |
| Max Gradient (%) | -8.7% | 8.7% | 2.0% |
| Net gradient (%) | -4.9% | 5.1% | 0.01% |
| Proportion > \| 4 \|%[3] | 0 | 100% | 5% |
| Proportion > + 4%[3] | 0 | 100% | 3% |
| Proportion < - 4%[3] | 0 | 100% | 2% |

*[1] Range C: 1440 to 4080m radius for design speed 120 km/h*
*[2] Range D: 720 to 1440m radius; Departure: <720m radius, for design speed 120 km/h.*
*[3] Desirable maximum gradient for dual carriageway*

## Traffic data

| Variable | Mini -mum | Maxi -mum | Mean |
|---|---|---|---|
| Average Annual Daily Flow | 3099 | 38785 | 13819 |
| AADT HGVs[1] | 339 | 7922 | 1617 |
| Per cent HGVs[1] | 4.4 | 27 | 12.4 |

*[1] Ignoring Census Points for which no HGV data were available*

## Link data - Continuous variables

| Variable | Mini -mum | Maxi -mum | Mean |
|---|---|---|---|
| Visibility (harmonic mean on link) (m) | 100 | 500 | 364 |
| Climbing lane length (total on link) (km) | 0 | 0 | 0.0 |
| Carriageway width (predominant on link) (m) | 7.3 | 11.0 | 7.5 |
| Number of lanes | 2 | 3 | 2.1 |
| ***On near-side:*** | | | |
| Number of accesses | 0 | 15 | 1.7 |
| Hardstrip width (mean on link) (m) | 0 | 2.0 | 0.7 |
| Verge width (mean on link) (m) | 0.5 | 10.0 | 2.4 |
| Obstruction severity (1-4) | 1 | 5 | 2.8 |
| Number of lighting columns | 0 | 30 | 1.1 |
| Min distance to lighting columns where present(m) | 0 | 9.9 | 1.6 |
| Number of other obstacles | 0 | 101 | 11.2 |
| Min distance to other obstacles where present(m) | 0.2 | 7.0 | 1.0 |
| Propn of link with obstruction | 0 | 50% | 0.8% |
| Propn of link with kerb | 0 | 100% | 55% |
| Propn of link with hardstrip | 0 | 100% | 69% |
| Propn of link with continuous edge line | 0 | 100% | 57% |
| ***On median side:*** | | | |
| Number of accesses | 0 | 11 | 1.0 |
| Hardstrip width (mean on link) (m) | 0 | 1.2 | 0.6 |
| Verge width (mean on link) (m) | 0.4 | 13.0 | 1.9 |
| Obstruction severity (1-4) | 1 | 5 | 3.0 |
| Number of lighting columns | 0 | 56 | 3.5 |
| Min distance to lighting columns where present(m) | 0.2 | 7.0 | 1.3 |
| Number of other obstacles | 0 | 46 | 3.4 |
| Min distance to other obstacles where present(m) | 0.0 | 5.0 | 1.0 |
| Propn of link with obstruction | 0 | 47% | 0.5% |
| Propn of link with kerb | 0 | 100% | 40% |
| Propn of link with hardstrip | 0 | 100% | 76% |
| Propn of link with continuous edge line | 0 | 100% | 54% |
| Propn of link with median safety fence | 0 | 100% | 50% |

## Appendix 4: List of variables measured (continued)

### Link data - Category variables

| *Variable* | *Category* | *Number of minor links* |
|---|---|---|
| Land Use | Urban | 80 |
| | Rural, mixed, other | 474 |
| Speed Limit | 70 mph | 533 |
| | Less than 70 mph | 21 |
| Climbing lane present | Yes | 0 |
| | No | 554 |
| Hard shoulder type | None | 219 |
| | Narrow hardstrip (1m or less) | 319 |
| | Full (2m) shoulder | 16 |
| Central reserve type | Safety fence | 275 |
| | Other barrier | 15 |
| | Open | 264 |
| Clearway signs | Present | 269 |
| | Not present | 285 |
| Road markings | None | 3 |
| | Short broken line | 524 |
| | Long broken line | 27 |
| | Other | 0 |

### Link data - Category variables for separate sides of carriageway

| | | *Number of minor links* | |
|---|---|---|---|
| *Variable* | *Category* | *Near-side* | *Off-side* |
| Signs on bend | Present | 7 | 23 |
| | Not present | 547 | 531 |
| Edge treatment | None | 18 | 55 |
| | Kerb only | 174 | 193 |
| | Kerb & hardstrip | 147 | 19 |
| | Kerb, hardstrip & French drain | 0 | 0 |
| | Hardstrip only | 146 | 266 |
| | Hardstrip & French drain | 58 | 18 |
| | Hardstrip & channel | 2 | 1 |
| | Lay-by | 6 | 1 |
| | Channel only | 2 | 0 |
| | Other | 1 | 1 |
| Edge markings | None | 107 | 114 |
| | Continuous white line | 429 | 427 |
| | Broken white line | 17 | 12 |
| | Other (including yellow line) | 1 | 1 |
| Verge type | Footpath/hard | 17 | 53 |
| | Grass | 0 | 0 |
| | Lay-by | 0 | 0 |
| | No verge | 529 | 500 |
| | Grass/hard then other roadway | 0 | 0 |
| | Other | 8 | 1 |
| Continuous obstruction | Safety fence | 60 | 279 |
| | Boundary fence | 35 | 1 |
| | Slope | 390 | 9 |
| | Bridge parapet | 7 | 0 |
| | Hedge/woods | 46 | 4 |
| | Retaining wall | 2 | 0 |
| | Rock face | 2 | 0 |
| | Tunnel | 0 | 0 |
| | Development | 3 | 0 |
| | Open median | 7 | 261 |
| | Other | 2 | 0 |
| Safety fences | Present | 404 | 379 |
| | Not present | 150 | 175 |
| Slope | Flat | 133 | 546 |
| | Embankment | 157 | 1 |
| | Cutting | 255 | 7 |
| | Rock face/wall | 1 | 0 |
| | Bridge parapet | 8 | 0 |
| Slope severity | None | 136 | 547 |
| | Very shallow (0-10 degrees) | 7 | 0 |
| | Shallow (10-30 degrees) | 28 | 1 |
| | Steep (>30 degrees) | 373 | 6 |
| | Rock face/wall | 2 | 0 |
| | Bridge parapet | 5 | 0 |
| Slope length | None | 146 | 547 |
| | Short (<4m) | 135 | 4 |
| | Medium (4-10m) | 163 | 2 |
| | Long (>10m) | 97 | 0 |
| | Rock face/wall | 2 | 0 |
| | Bridge parapet | 8 | 0 |
| | Other/not visible | 3 | 1 |
| Lighting columns | Present | 120 | 176 |
| | Not present | 434 | 378 |

# Appendix 4: List of variables measured (continued)

## Major junction data - Continuous variables

| *Variable* | *Minimum* | *Maximum* | *Mean* |
|---|---|---|---|
| Number of junction arms | 2 | 6 | 4.3 |
| Roundabout island diam (m) | 15 | 90 | 43.6 |
| ***For each junction arm:*** | | | |
| Number of lanes at stop line | 1 | 4 | 2.1 |
| Entry width (m) | 3.0 | 20.0 | 8.0 |
| Entry flare length (m) | 0 | 100 | 597 |
| Exit width (m) | 0 | 12.0 | 6.1 |

## Major junction data - Category variables

| *Variable* | *Category* | *Number of major junctions* |
|---|---|---|
| Major junction type | Roundabout | 53 |
| | Roundabout with partial signals | 2 |
| | Roundabout with full signals | 0 |
| | Traffic Signals | 2 |
| Lighting | Present | 55 |
| | Not present | 2 |
| Roundabout island type | Not a roundabout | 2 |
| | Grass | 27 |
| | Concrete | 0 |
| | Grass & shrubs | 22 |
| | Mixed hard/planting | 6 |
| | Other | 0 |
| Yellow box markings | No yellow box/KEEP CLEAR marks | 54 |
| | Yellow boxes | 0 |
| | KEEP CLEAR marks | 3 |
| | Mixture | 0 |

## Major junction data - Category variables for individual junction arms

| *Variable* | *Category* | *Number of major junction arms* |
|---|---|---|
| Road markings[1] | None | 64 |
| | Arrows | 22 |
| | Route plus arrows | 5 |
| | Speed bars | 19 |
| | SLOW | 6 |
| | STOP | 0 |
| | Other | 4 |
| | Not visible | 131 |
| Left turn lane | None | 233 |
| | Painted lines | 7 |
| | Physical separation | 0 |
| | Not visible | 2 |
| | Island but no lane | 1 |
| Signs on roundabout island | Not a roundabout | 8 |
| | None | 0 |
| | KEEP LEFT arrow | 1 |
| | Chevrons | 3 |
| | KEEP LEFT and chevrons | 115 |
| | Other or not visible | 116 |
| Signals at entry to roundabout | None | 237 |
| | Part-time | 6 |
| | Full-time | 0 |
| Signal gantry | No signals | 238 |
| | Signals but no gantry | 5 |
| | Gantry present | 0 |
| Pedestrian/cycle facilities | None | 209 |
| | Crossing, no signals | 2 |
| | Crossing with signals | 1 |
| | Bridge or underpass | 10 |
| | Drop kerbs | 6 |
| | Refuge | 15 |
| | Cycle lane | 0 |
| | Mixed | 0 |
| | Other | 0 |

*[1] Sum of categories is greater than number of junction arms because some junction arms have more than one marking*

# Appendix 4: List of variables measured (continued)

## Minor junction data - Continuous variables

| *Variable* | *Minimum* | *Maximum* | *Mean* |
|---|---|---|---|
| ***Number of entries/exits:*** | | | |
| Nearside | 0 | 3 | 1.3 |
| Median side | 0 | 3 | 0.3 |
| Length of junction (m) | 0 | 5195 | 453 |
| Distance between exit and entry (m) | 0 | 2070 | 181 |
| Length of stagger, where present (m) | 0 | 480 | 144 |
| Length of median gap, where present (m) | 0 | 60 | 16 |
| ***For each junction arm:*** | | | |
| Number of lanes on exit/entry | 1 | 4 | 1.8 |
| Length of deceleration lane (m) | 0 | 525 | 68 |
| Length of acceleration lane (m) | 0 | 525 | 78 |
| Number of arms with island | 0 | 3 | 0.2 |

## Minor junction data - Category variables

| *Variable* | *Category* | *Number of minor junctions* |
|---|---|---|
| Junction type | Off/on slips | 89 |
| | T-junc with large island | 34 |
| | T-junc with small/no island | 29 |
| | Exit only | 35 |
| | Entry only | 34 |
| | T-junc, crossover from opp c/way | 15 |
| | Crossover to T-junc on opp c/way | 16 |
| | Staggered junction with crossover | 20 |
| | Staggered with temporary barrier | 0 |
| | Staggered with permanent barrier | 0 |
| | Crossroads | 7 |
| | Scheme leaves other road | 1 |
| | Scheme joins other road | 0 |
| | Other type | 11 |
| | Other type with accesses | 13 |
| | Not known | 31 |
| Lighting | Present | 50 |
| | Not present | 285 |
| Road markings on main carriageway[1] | None | 155 |
| | Arrows | 173 |
| | Route plus arrows | 6 |
| | Speed bars | 2 |
| | SLOW | 2 |
| | Other | 0 |

[1] *Sum of categories is greater than number of junctions because some junctions have more than one marking*

## Minor junction data - Category variables for individual junction arms

| *Variable* | *Category* | *Number of Minor junction arms* |
|---|---|---|
| Road markings in exit/entry[1] | None | 241 |
| | Arrows | 66 |
| | Route plus arrows | 6 |
| | Speed bars | 0 |
| | SLOW | 2 |
| | STOP | 0 |
| | Other | 71 |
| | Not visible | 181 |
| Pedestrian/cycle facilities | None | 527 |
| | Crossing, no signals | 19 |
| | Crossing with signals | 0 |
| | Bridge or underpass | 4 |
| | Drop kerbs | 4 |
| | Refuge | 4 |
| | Cycle lane | 0 |
| | Mixed | 0 |
| | Other | 0 |

[1] *Sum of categories is greater than number of junction arms because some junction arms have more than one marking*

# Appendix 5: Accident rates, by scheme, 1988-92

| | | | | | | | | | | | | | | | Number of accidents | | | | | Accident rates per MVkm | | | Junc rates per Mveh | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| *Scheme* | *Kerb* | *Strip* | *Fence* | *Age band* | *Acc yrs* | *Carr width* | *Total length km* | *Link length km* | *MjLink length km* | *Total MVkm* | *Link MVkm* | *MjLink MVkm* | *MjJunc AADT* | *MnJunc AADT* | *Total* | *Link* | *MjJnc* | *MnJnc* | *MjLink* | *Link* | *MjLink* | *Scheme* | *MjJnc* | *MnJnc* |
| 1 | 0 | 1 | 1 | OLD | 5 | D2 | 28.4 | 21.0 | 28.4 | 701.7 | 520.0 | 701.7 | 0.0 | 533.8 | 58 | 47 | 0 | 11 | 58 | 0.090 | 0.083 | 0.083 | 0.000 | 0.056 |
| 3 | 0 | 1 | 0 | OLD | 5 | D3 | 25.7 | 23.8 | 25.7 | 1039.1 | 962.0 | 1039.1 | 0.0 | 542.9 | 58 | 52 | 0 | 6 | 58 | 0.054 | 0.056 | 0.056 | 0.000 | 0.030 |
| 6 | 0 | 1 | 0 | OLD | 5 | D2 | 13.9 | 13.0 | 13.9 | 540.5 | 503.6 | 540.5 | 0.0 | 212.7 | 56 | 46 | 0 | 10 | 56 | 0.091 | 0.104 | 0.104 | 0.000 | 0.129 |
| 7 | 1 | 0 | 1 | OLD | 5 | D3 | 10.2 | 6.3 | 10.2 | 530.8 | 332.0 | 530.8 | 0.0 | 927.9 | 70 | 49 | 0 | 21 | 70 | 0.148 | 0.132 | 0.132 | 0.000 | 0.062 |
| 8 | 0 | 1 | 0 | MID | 5 | D2 | 22.9 | 18.7 | 22.9 | 351.3 | 300.1 | 351.3 | 0.0 | 253.0 | 51 | 38 | 0 | 13 | 51 | 0.127 | 0.145 | 0.145 | 0.000 | 0.141 |
| 9 | 0 | 1 | 0 | OLD | 5 | D2 | 13.7 | 9.8 | 13.7 | 500.1 | 358.5 | 500.1 | 0.0 | 400.1 | 58 | 39 | 0 | 19 | 58 | 0.109 | 0.116 | 0.116 | 0.000 | 0.130 |
| 10 | 0 | 1 | 0 | OLD | 5 | D2 | 14.3 | 7.8 | 14.3 | 357.2 | 189.9 | 357.2 | 0.0 | 522.1 | 73 | 49 | 0 | 24 | 73 | 0.258 | 0.204 | 0.204 | 0.000 | 0.126 |
| 11 | 1 | 0 | 0 | OLD | 5 | D2 | 7.9 | 6.8 | 7.2 | 335.5 | 288.5 | 302.7 | 947.3 | 777.2 | 166 | 51 | 109 | 6 | 57 | 0.177 | 0.188 | 0.495 | 0.315 | 0.021 |
| 12 | 1 | 1 | 1 | OLD | 5 | D2 | 5.9 | 3.5 | 5.0 | 122.9 | 73.6 | 105.0 | 226.7 | 113.4 | 74 | 28 | 38 | 8 | 36 | 0.381 | 0.343 | 0.602 | 0.459 | 0.193 |
| 13 | 0 | 1 | 0 | OLD | 5 | D2 | 6.7 | 4.8 | 6.7 | 72.4 | 52.1 | 72.4 | 0.0 | 118.1 | 15 | 9 | 0 | 6 | 15 | 0.173 | 0.207 | 0.207 | 0.000 | 0.139 |
| 14 | 0 | 1 | 1 | NEW | 5 | D2 | 8.1 | 6.9 | 7.9 | 153.2 | 130.4 | 149.8 | 104.1 | 208.3 | 50 | 19 | 8 | 23 | 42 | 0.146 | 0.280 | 0.326 | 0.210 | 0.303 |
| 15 | 0 | 1 | 0 | OLD | 4 | D2 | 13.1 | 10.0 | 13.1 | 128.1 | 98.4 | 128.1 | 0.0 | 107.6 | 13 | 6 | 0 | 7 | 13 | 0.061 | 0.101 | 0.101 | 0.000 | 0.178 |
| 16 | 1 | 0 | 1 | MID | 5 | D2 | 10.9 | 10.3 | 10.4 | 253.4 | 239.5 | 241.9 | 509.8 | 127.4 | 44 | 19 | 14 | 11 | 30 | 0.079 | 0.124 | 0.174 | 0.075 | 0.236 |
| 17 | 0 | 1 | 1 | MID | 5 | D2 | 5.1 | 4.8 | 4.8 | 126.9 | 120.4 | 120.4 | 272.6 | 0.0 | 22 | 9 | 8 | 5 | 14 | 0.075 | 0.116 | 0.173 | 0.080 | 0.000 |
| 18 | 1 | 1 | 0 | OLD | 5 | D2 | 19.6 | 14.1 | 19.6 | 556.0 | 400.9 | 556.0 | 0.0 | 622.3 | 52 | 40 | 0 | 12 | 52 | 0.100 | 0.094 | 0.094 | 0.000 | 0.053 |
| 19 | 0 | 1 | 0 | OLD | 5 | D2 | 14.2 | 10.9 | 14.2 | 321.4 | 247.8 | 321.4 | 0.0 | 373.1 | 25 | 17 | 0 | 8 | 25 | 0.069 | 0.078 | 0.078 | 0.000 | 0.059 |
| 20 | 0 | 1 | 0 | MID | 5 | D2 | 12.3 | 10.6 | 12.3 | 382.0 | 309.2 | 382.0 | 0.0 | 223.9 | 28 | 26 | 0 | 2 | 28 | 0.084 | 0.073 | 0.073 | 0.000 | 0.024 |
| 21 | 1 | 0 | 1 | MID | 5 | D2 | 4.8 | 4.3 | 4.8 | 195.9 | 173.4 | 195.9 | 0.0 | 445.4 | 27 | 17 | 0 | 10 | 27 | 0.098 | 0.138 | 0.138 | 0.000 | 0.062 |
| 22 | 1 | 1 | 1 | OLD | 5 | D2 | 11.3 | 8.8 | 10.7 | 446.7 | 345.8 | 423.9 | 442.8 | 979.3 | 45 | 22 | 6 | 17 | 39 | 0.064 | 0.092 | 0.101 | 0.037 | 0.048 |
| 23 | 1 | 0 | 0 | OLD | 5 | D2 | 3.1 | 3.0 | 3.1 | 105.3 | 100.2 | 105.3 | 0.0 | 93.0 | 8 | 7 | 0 | 1 | 8 | 0.070 | 0.076 | 0.076 | 0.000 | 0.029 |
| 24 | 1 | 0 | 0 | OLD | 5 | D2 | 2.3 | 2.0 | 2.3 | 76.7 | 66.5 | 76.7 | 0.0 | 372.1 | 18 | 9 | 0 | 9 | 18 | 0.135 | 0.235 | 0.235 | 0.000 | 0.066 |
| 26 | 1 | 1 | 0 | OLD | 5 | D2 | 8.9 | 8.0 | 8.9 | 265.8 | 238.3 | 265.8 | 0.0 | 164.0 | 33 | 30 | 0 | 3 | 33 | 0.126 | 0.124 | 0.124 | 0.000 | 0.050 |
| 27 | 1 | 1 | 1 | OLD | 5 | D2 | 16.3 | 11.8 | 16.3 | 458.3 | 331.5 | 458.3 | 0.0 | 616.5 | 81 | 59 | 0 | 22 | 81 | 0.178 | 0.177 | 0.177 | 0.000 | 0.098 |
| 28 | 1 | 1 | 1 | MID | 5 | D2 | 12.5 | 8.0 | 10.8 | 363.7 | 230.8 | 314.6 | 479.0 | 319.1 | 79 | 27 | 48 | 4 | 31 | 0.117 | 0.099 | 0.217 | 0.275 | 0.034 |
| 29 | 1 | 0 | 1 | OLD | 5 | D2 | 4.5 | 3.5 | 3.5 | 150.8 | 118.7 | 118.7 | 183.7 | 0.0 | 45 | 12 | 23 | 10 | 22 | 0.101 | 0.185 | 0.298 | 0.343 | 0.000 |
| 30 | 1 | 1 | 1 | OLD | 5 | D2 | 12.6 | 12.5 | 12.5 | 232.4 | 229.8 | 229.8 | 100.9 | 0.0 | 22 | 18 | 4 | 0 | 18 | 0.078 | 0.078 | 0.095 | 0.109 | 0.000 |
| 32 | 1 | 0 | 0 | OLD | 5 | D2 | 9.1 | 8.1 | 8.8 | 154.1 | 137.3 | 150.4 | 93.3 | 139.9 | 22 | 4 | 17 | 1 | 5 | 0.029 | 0.033 | 0.143 | 0.499 | 0.020 |
| 33 | 0 | 0 | 0 | OLD | 5 | D2 | 7.1 | 6.0 | 7.1 | 185.8 | 158.2 | 185.8 | 0.0 | 287.9 | 11 | 9 | 0 | 2 | 11 | 0.057 | 0.059 | 0.059 | 0.000 | 0.019 |
| 34 | 1 | 1 | 1 | MID | 5 | D2 | 10.9 | 10.3 | 10.3 | 183.0 | 172.9 | 172.9 | 276.6 | 0.0 | 42 | 17 | 24 | 1 | 18 | 0.098 | 0.104 | 0.229 | 0.238 | 0.000 |
| 35 | 0 | 1 | 1 | NEW | 4 | D2 | 9.8 | 9.0 | 9.6 | 112.5 | 103.3 | 109.9 | 141.9 | 202.6 | 11 | 5 | 4 | 2 | 7 | 0.048 | 0.064 | 0.098 | 0.077 | 0.027 |
| 36 | 0 | 1 | 1 | OLD | 5 | D2 | 5.4 | 3.9 | 5.4 | 107.4 | 78.4 | 107.4 | 0.0 | 219.2 | 21 | 14 | 0 | 7 | 21 | 0.179 | 0.196 | 0.196 | 0.000 | 0.087 |
| 37 | 1 | 0 | 1 | OLD | 5 | D2 | 7.8 | 7.1 | 7.2 | 285.8 | 263.5 | 266.1 | 317.9 | 143.2 | 60 | 32 | 17 | 11 | 43 | 0.121 | 0.162 | 0.210 | 0.147 | 0.210 |
| 39 | 1 | 0 | 1 | OLD | 5 | D2 | 9.5 | 7.7 | 9.5 | 156.8 | 128.2 | 156.8 | 0.0 | 298.5 | 55 | 37 | 0 | 18 | 55 | 0.289 | 0.351 | 0.351 | 0.000 | 0.165 |
| 40 | 1 | 0 | 1 | OLD | 5 | D2 | 5.0 | 3.0 | 5.0 | 107.1 | 63.9 | 107.1 | 0.0 | 117.2 | 7 | 6 | 0 | 1 | 7 | 0.094 | 0.065 | 0.065 | 0.000 | 0.023 |
| 42 | 1 | 0 | 1 | OLD | 5 | D2 | 3.8 | 3.4 | 3.8 | 81.3 | 72.7 | 81.3 | 0.0 | 234.3 | 19 | 14 | 0 | 5 | 19 | 0.193 | 0.234 | 0.234 | 0.000 | 0.058 |
| 43 | 1 | 0 | 1 | OLD | 5 | D2 | 3.1 | 2.2 | 3.1 | 66.1 | 46.4 | 66.1 | 0.0 | 234.3 | 14 | 6 | 0 | 8 | 14 | 0.129 | 0.212 | 0.212 | 0.000 | 0.094 |
| 44 | 1 | 0 | 1 | OLD | 5 | D2 | 8.2 | 7.9 | 8.2 | 144.9 | 139.5 | 144.9 | 0.0 | 118.3 | 16 | 12 | 0 | 4 | 16 | 0.086 | 0.110 | 0.110 | 0.000 | 0.093 |
| 45 | 1 | 0 | 1 | OLD | 5 | D2 | 14.8 | 14.0 | 14.8 | 176.5 | 164.6 | 176.5 | 0.0 | 166.8 | 35 | 34 | 0 | 1 | 35 | 0.207 | 0.198 | 0.198 | 0.000 | 0.016 |
| 46 | 0 | 1 | 1 | OLD | 5 | D2 | 25.3 | 21.2 | 24.6 | 367.3 | 306.7 | 355.7 | 170.4 | 204.9 | 69 | 33 | 17 | 19 | 52 | 0.108 | 0.146 | 0.188 | 0.273 | 0.254 |
| 53 | 1 | 1 | 1 | OLD | 5 | D2 | 15.5 | 13.8 | 15.5 | 513.4 | 456.8 | 513.4 | 0.0 | 908.8 | 68 | 58 | 0 | 10 | 68 | 0.127 | 0.132 | 0.132 | 0.000 | 0.030 |
| 54 | 1 | 1 | 0 | OLD | 5 | D2 | 6.9 | 6.1 | 6.9 | 77.4 | 68.6 | 77.4 | 0.0 | 61.4 | 11 | 9 | 0 | 2 | 11 | 0.131 | 0.142 | 0.142 | 0.000 | 0.089 |

| Scheme | Kerb | Strip | Fence | Age band | Acc yrs | Carr width | Total length km | Link length km | MjLink length km | Total MVkm | Link MVkm | MjLink MVkm | MjJunc AADT | MnJunc AADT | Number of accidents | | | | | Accident rates per MVkm | | | Junc rates per Mveh | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | | | | Total | Link | MjJnc | MnJnc | MjLink | Link | MjLink | Scheme | MjJnc | MnJnc |
| 58 | 1 | 1 | 0 | OLD | 5 | D2 | 1.7 | 1.6 | 1.7 | 9.7 | 9.0 | 9.7 | 0.0 | 15.5 | 1 | 0 | 0 | 1 | 1 | 0.000 | 0.104 | 0.104 | 0.000 | 0.176 |
| 59 | 0 | 1 | 0 | OLD | 5 | D2 | 6.7 | 6.6 | 6.7 | 51.4 | 50.6 | 51.4 | 0.0 | 42.1 | 9 | 8 | 0 | 1 | 9 | 0.158 | 0.175 | 0.175 | 0.000 | 0.065 |
| 60 | 0 | 1 | 0 | MID | 2 | D2 | 12.4 | 11.3 | 12.4 | 39.4 | 36.1 | 39.4 | 0.0 | 34.9 | 1 | 1 | 0 | 0 | 1 | 0.028 | 0.025 | 0.025 | 0.000 | 0.000 |
| 61 | 1 | 0 | 0 | OLD | 5 | D2 | 4.3 | 4.3 | 4.3 | 178.5 | 178.5 | 178.5 | 0.0 | 0.0 | 7 | 7 | 0 | 0 | 7 | 0.039 | 0.039 | 0.039 | 0.000 | 0.000 |
| 64 | 1 | 0 | 1 | OLD | 5 | D2 | 3.7 | 3.3 | 3.4 | 127.4 | 110.1 | 115.4 | 695.3 | 268.5 | 43 | 12 | 26 | 5 | 17 | 0.109 | 0.147 | 0.338 | 0.102 | 0.051 |
| 65 | 0 | 1 | 0 | OLD | 5 | D2 | 11.9 | 10.8 | 11.6 | 312.7 | 284.2 | 304.8 | 144.0 | 287.9 | 91 | 39 | 43 | 9 | 48 | 0.137 | 0.157 | 0.291 | 0.818 | 0.086 |
| 66 | 0 | 1 | 0 | NEW | 3 | D2 | 8.4 | 7.0 | 8.4 | 127.0 | 105.1 | 127.0 | 0.0 | 206.2 | 8 | 7 | 0 | 1 | 8 | 0.067 | 0.063 | 0.063 | 0.000 | 0.013 |
| 67 | 1 | 1 | 0 | OLD | 5 | D2 | 3.1 | 1.3 | 3.1 | 122.1 | 49.3 | 122.1 | 0.0 | 214.5 | 20 | 8 | 0 | 12 | 20 | 0.162 | 0.164 | 0.164 | 0.000 | 0.153 |
| 68 | 0 | 1 | 0 | OLD | 5 | D2 | 7.3 | 6.9 | 6.9 | 173.1 | 163.6 | 163.6 | 130.7 | 0.0 | 21 | 14 | 4 | 3 | 17 | 0.086 | 0.104 | 0.121 | 0.084 | 0.000 |
| 69 | 1 | 0 | 0 | OLD | 5 | D2 | 10.4 | 8.5 | 10.0 | 315.8 | 258.0 | 301.9 | 165.7 | 331.5 | 52 | 39 | 8 | 5 | 44 | 0.151 | 0.146 | 0.165 | 0.132 | 0.041 |
| 71 | 0 | 1 | 1 | NEW | 4 | D2 | 24.8 | 24.2 | 24.8 | 559.5 | 546.0 | 559.5 | 0.0 | 123.5 | 34 | 33 | 0 | 1 | 34 | 0.060 | 0.061 | 0.061 | 0.000 | 0.022 |
| 73 | 0 | 1 | 1 | MID | 5 | D2 | 11.8 | 8.8 | 11.8 | 171.6 | 127.1 | 171.6 | 0.0 | 318.1 | 25 | 12 | 0 | 13 | 25 | 0.094 | 0.146 | 0.146 | 0.000 | 0.112 |
| 74 | 1 | 0 | 1 | OLD | 5 | D2 | 11.6 | 7.7 | 11.6 | 248.4 | 164.5 | 248.4 | 0.0 | 355.0 | 22 | 16 | 0 | 6 | 22 | 0.097 | 0.089 | 0.089 | 0.000 | 0.046 |
| 76 | 0 | 1 | 0 | OLD | 5 | D2 | 16.3 | 15.1 | 16.3 | 294.4 | 272.9 | 294.4 | 0.0 | 197.9 | 20 | 19 | 0 | 1 | 20 | 0.070 | 0.068 | 0.068 | 0.000 | 0.014 |
| 77 | 0 | 1 | 0 | MID | 5 | D2 | 17.9 | 15.7 | 17.9 | 412.5 | 361.5 | 412.5 | 0.0 | 252.2 | 48 | 38 | 0 | 10 | 48 | 0.105 | 0.116 | 0.116 | 0.000 | 0.109 |
| 80 | 1 | 1 | 1 | OLD | 5 | D3 | 19.2 | 16.2 | 19.2 | 1297.3 | 1090.2 | 1297.3 | 0.0 | 2241.1 | 234 | 160 | 0 | 74 | 234 | 0.147 | 0.180 | 0.180 | 0.000 | 0.090 |
| 81 | 1 | 0 | 0 | OLD | 5 | D2 | 5.8 | 4.3 | 5.8 | 141.3 | 105.3 | 141.3 | 0.0 | 133.0 | 24 | 18 | 0 | 6 | 24 | 0.171 | 0.170 | 0.170 | 0.000 | 0.124 |
| 82 | 0 | 1 | 0 | MID | 5 | D2 | 17.1 | 14.3 | 17.1 | 229.2 | 188.5 | 229.2 | 0.0 | 329.9 | 35 | 28 | 0 | 7 | 35 | 0.149 | 0.153 | 0.153 | 0.000 | 0.058 |
| 84 | 1 | 0 | 0 | OLD | 5 | D2 | 9.5 | 6.5 | 9.5 | 369.6 | 255.5 | 369.6 | 0.0 | 856.3 | 45 | 22 | 0 | 23 | 45 | 0.086 | 0.122 | 0.122 | 0.000 | 0.074 |
| 85 | 1 | 1 | 1 | OLD | 5 | D3 | 23.1 | 18.8 | 23.1 | 1308.0 | 1072.9 | 1308.0 | 0.0 | 930.2 | 151 | 114 | 0 | 37 | 151 | 0.106 | 0.115 | 0.115 | 0.000 | 0.109 |
| 86 | 1 | 0 | 1 | MID | 5 | D2 | 16.0 | 14.0 | 16.0 | 794.0 | 684.8 | 794.0 | 0.0 | 1397.0 | 172 | 89 | 0 | 83 | 172 | 0.130 | 0.217 | 0.217 | 0.000 | 0.163 |
| 87 | 0 | 1 | 1 | OLD | 5 | D3 | 11.4 | 10.7 | 11.4 | 739.2 | 688.2 | 739.2 | 0.0 | 708.1 | 61 | 48 | 0 | 13 | 61 | 0.070 | 0.083 | 0.083 | 0.000 | 0.050 |
| 91 | 0 | 1 | 0 | OLD | 5 | D2 | 22.2 | 16.9 | 22.2 | 388.4 | 295.8 | 388.4 | 0.0 | 387.8 | 48 | 29 | 0 | 19 | 48 | 0.098 | 0.124 | 0.124 | 0.000 | 0.134 |
| 93 | 0 | 1 | 0 | OLD | 5 | D2 | 5.4 | 4.9 | 5.4 | 63.0 | 55.5 | 63.0 | 0.0 | 157.8 | 8 | 8 | 0 | 0 | 8 | 0.144 | 0.127 | 0.127 | 0.000 | 0.000 |
| 94 | 0 | 1 | 0 | OLD | 5 | D2 | 6.5 | 5.8 | 6.5 | 114.6 | 103.4 | 114.6 | 0.0 | 388.6 | 24 | 13 | 0 | 11 | 24 | 0.126 | 0.209 | 0.209 | 0.000 | 0.078 |
| 95 | 0 | 1 | 1 | NEW | 4 | D2 | 4.9 | 4.9 | 4.9 | 87.0 | 87.0 | 87.0 | 0.0 | 0.0 | 9 | 9 | 0 | 0 | 9 | 0.103 | 0.103 | 0.103 | 0.000 | 0.000 |
| 96 | 0 | 1 | 0 | OLD | 5 | D2 | 11.1 | 9.4 | 11.1 | 356.4 | 303.1 | 356.4 | 0.0 | 351.9 | 25 | 16 | 0 | 9 | 25 | 0.053 | 0.070 | 0.070 | 0.000 | 0.070 |
| 97 | 0 | 1 | 0 | OLD | 5 | D2 | 23.8 | 23.4 | 23.8 | 226.9 | 223.1 | 226.9 | 0.0 | 104.7 | 23 | 18 | 0 | 5 | 23 | 0.081 | 0.101 | 0.101 | 0.000 | 0.131 |
| 98 | 0 | 1 | 0 | OLD | 5 | D2 | 17.6 | 17.4 | 17.6 | 168.1 | 165.7 | 168.1 | 0.0 | 52.3 | 11 | 10 | 0 | 1 | 11 | 0.060 | 0.065 | 0.065 | 0.000 | 0.052 |
| 99 | 0 | 1 | 1 | NEW | 5 | D2 | 16.8 | 16.6 | 16.6 | 160.5 | 158.4 | 158.4 | 52.3 | 0.0 | 20 | 17 | 3 | 0 | 17 | 0.107 | 0.107 | 0.125 | 0.157 | 0.000 |
| 100 | 0 | 1 | 0 | MID | 5 | D2 | 20.7 | 19.2 | 20.5 | 263.5 | 244.9 | 260.6 | 71.7 | 141.0 | 19 | 9 | 9 | 1 | 10 | 0.037 | 0.038 | 0.072 | 0.344 | 0.019 |
| 101 | 0 | 1 | 1 | OLD | 5 | D2 | 8.6 | 7.1 | 8.6 | 217.7 | 179.8 | 217.7 | 0.0 | 276.8 | 19 | 12 | 0 | 7 | 19 | 0.067 | 0.087 | 0.087 | 0.000 | 0.069 |
| 102 | 0 | 1 | 1 | OLD | 5 | D2 | 14.0 | 12.3 | 14.0 | 454.9 | 398.4 | 454.9 | 0.0 | 356.1 | 23 | 20 | 0 | 3 | 23 | 0.050 | 0.051 | 0.051 | 0.000 | 0.023 |
| 103 | 0 | 1 | 1 | OLD | 5 | D2 | 5.2 | 3.8 | 5.2 | 168.3 | 124.1 | 168.3 | 0.0 | 356.1 | 13 | 6 | 0 | 7 | 13 | 0.048 | 0.077 | 0.077 | 0.000 | 0.054 |
| 104 | 0 | 1 | 1 | MID | 5 | D2 | 9.7 | 5.9 | 8.8 | 341.6 | 193.8 | 291.6 | 285.6 | 384.4 | 150 | 31 | 60 | 59 | 90 | 0.160 | 0.309 | 0.439 | 0.576 | 0.420 |
| 105 | 1 | 0 | 1 | OLD | 5 | D2 | 6.4 | 6.0 | 6.0 | 133.9 | 125.6 | 125.6 | 229.4 | 0.0 | 28 | 12 | 15 | 1 | 13 | 0.096 | 0.104 | 0.209 | 0.179 | 0.000 |
| 106 | 0 | 1 | 0 | OLD | 5 | D2 | 14.6 | 14.0 | 14.6 | 193.4 | 185.7 | 193.2 | 71.7 | 73.7 | 23 | 7 | 13 | 3 | 10 | 0.038 | 0.052 | 0.119 | 0.497 | 0.112 |
| 107 | 0 | 1 | 1 | NEW | 3 | D2 | 10.4 | 10.0 | 10.3 | 80.2 | 77.2 | 79.5 | 40.0 | 62.7 | 13 | 7 | 5 | 1 | 8 | 0.091 | 0.101 | 0.162 | 0.342 | 0.044 |
| 109 | 0 | 0 | 0 | OLD | 5 | D2 | 6.0 | 5.4 | 6.0 | 75.3 | 68.2 | 75.3 | 0.0 | 137.5 | 7 | 6 | 0 | 1 | 7 | 0.088 | 0.093 | 0.093 | 0.000 | 0.020 |
| 110 | 1 | 0 | 1 | OLD | 5 | D2 | 6.4 | 5.8 | 5.8 | 90.6 | 82.7 | 82.7 | 77.4 | 0.0 | 27 | 13 | 11 | 3 | 16 | 0.157 | 0.193 | 0.298 | 0.389 | 0.000 |
| 111 | 1 | 0 | 0 | OLD | 5 | D2 | 14.8 | 13.2 | 14.8 | 144.1 | 127.9 | 144.1 | 0.0 | 112.4 | 33 | 32 | 0 | 1 | 33 | 0.250 | 0.229 | 0.229 | 0.000 | 0.024 |
| 112 | 1 | 0 | 0 | OLD | 5 | D2 | 9.8 | 8.7 | 9.8 | 142.1 | 125.6 | 142.1 | 0.0 | 79.1 | 17 | 15 | 0 | 2 | 17 | 0.119 | 0.120 | 0.120 | 0.000 | 0.069 |
| 113 | 1 | 1 | 1 | NEW | 3 | D2 | 3.3 | 2.7 | 2.7 | 30.3 | 24.8 | 24.8 | 100.8 | 0.0 | 5 | 0 | 5 | 0 | 0 | 0.000 | 0.000 | 0.165 | 0.136 | 0.000 |
| 114 | 1 | 0 | 0 | OLD | 5 | D2 | 8.9 | 7.0 | 8.9 | 217.9 | 171.1 | 217.9 | 0.0 | 340.4 | 22 | 7 | 0 | 15 | 22 | 0.041 | 0.101 | 0.101 | 0.000 | 0.121 |

| Scheme | Kerb | Strip | Fence | Age band | Acc yrs | Carr width | Total length km | Link length km | MjLink length km | Total MVkm | Link MVkm | MjLink MVkm | MjJunc AADT | MnJunc AADT | Number of accidents: Total | Link | MjJnc | MnJnc | MjLink | Accident rates per MVkm: Link | MjLink | Scheme | Junc rates per Mveh: MjJnc | MnJnc |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 115 | 1 | 0 | 0 | OLD | 5 | D2 | 10.1 | 9.5 | 9.7 | 283.2 | 267.6 | 271.9 | 163.7 | 163.7 | 54 | 29 | 20 | 5 | 34 | 0.108 | 0.125 | 0.191 | 0.335 | 0.084 |
| 117 | 1 | 0 | 0 | OLD | 5 | D2 | 4.4 | 3.2 | 4.4 | 69.3 | 51.0 | 69.3 | 0.0 | 205.5 | 12 | 3 | 0 | 9 | 12 | 0.059 | 0.173 | 0.173 | 0.000 | 0.120 |
| 118 | 1 | 0 | 0 | OLD | 5 | D2 | 25.0 | 22.6 | 24.8 | 426.0 | 386.2 | 423.4 | 88.5 | 975.6 | 60 | 33 | 7 | 20 | 53 | 0.085 | 0.125 | 0.141 | 0.217 | 0.056 |
| 121 | 1 | 0 | 0 | OLD | 5 | D2 | 7.2 | 6.2 | 6.6 | 177.1 | 153.7 | 162.3 | 270.3 | 270.3 | 26 | 14 | 4 | 8 | 22 | 0.091 | 0.136 | 0.147 | 0.041 | 0.081 |
| 122 | 1 | 1 | 1 | OLD | 5 | D2 | 12.8 | 12.8 | 12.8 | 199.3 | 198.7 | 198.7 | 85.2 | 0.0 | 32 | 18 | 10 | 4 | 22 | 0.091 | 0.111 | 0.161 | 0.322 | 0.000 |
| 123 | 1 | 1 | 1 | MID | 5 | D2 | 18.6 | 18.6 | 18.6 | 325.2 | 325.2 | 325.2 | 0.0 | 0.0 | 28 | 28 | 0 | 0 | 28 | 0.086 | 0.086 | 0.086 | 0.000 | 0.000 |
| 124 | 1 | 1 | 1 | MID | 5 | D2 | 24.8 | 21.0 | 24.8 | 457.1 | 382.5 | 457.1 | 0.0 | 307.0 | 65 | 41 | 0 | 24 | 65 | 0.107 | 0.142 | 0.142 | 0.000 | 0.214 |
| 125 | 0 | 1 | 1 | OLD | 5 | D2 | 2.3 | 1.1 | 2.3 | 61.4 | 28.2 | 61.4 | 0.0 | 300.3 | 26 | 17 | 0 | 9 | 26 | 0.602 | 0.424 | 0.424 | 0.000 | 0.082 |
| 126 | 1 | 1 | 1 | OLD | 5 | D2 | 4.6 | 3.2 | 4.6 | 65.1 | 44.3 | 65.1 | 0.0 | 244.3 | 11 | 9 | 0 | 2 | 11 | 0.203 | 0.169 | 0.169 | 0.000 | 0.022 |
| 127 | 1 | 0 | 1 | OLD | 5 | D2 | 17.5 | 16.2 | 17.5 | 335.3 | 312.6 | 335.3 | 0.0 | 514.4 | 29 | 17 | 0 | 12 | 29 | 0.054 | 0.086 | 0.086 | 0.000 | 0.064 |
| 128 | 0 | 1 | 1 | OLD | 5 | D2 | 25.7 | 20.7 | 25.4 | 535.0 | 431.7 | 525.6 | 171.6 | 355.8 | 63 | 31 | 15 | 17 | 48 | 0.072 | 0.091 | 0.118 | 0.240 | 0.131 |
| 129 | 0 | 1 | 0 | OLD | 5 | D2 | 12.4 | 11.4 | 12.4 | 130.1 | 119.9 | 130.1 | 0.0 | 167.9 | 21 | 15 | 0 | 6 | 21 | 0.125 | 0.161 | 0.161 | 0.000 | 0.098 |
| 130 | 0 | 1 | 1 | NEW | 4 | D2 | 8.0 | 6.8 | 8.0 | 240.7 | 204.9 | 240.7 | 0.0 | 656.1 | 24 | 15 | 0 | 9 | 24 | 0.073 | 0.100 | 0.100 | 0.000 | 0.038 |
| 131 | 0 | 1 | 1 | NEW | 4 | D2 | 8.0 | 7.8 | 7.8 | 153.0 | 149.6 | 149.6 | 105.3 | 0.0 | 18 | 16 | 2 | 0 | 16 | 0.107 | 0.107 | 0.118 | 0.052 | 0.000 |
| 132 | 1 | 1 | 1 | NEW | 4 | D2 | 6.3 | 5.7 | 5.9 | 79.8 | 72.7 | 75.3 | 139.4 | 69.7 | 34 | 9 | 23 | 2 | 11 | 0.124 | 0.146 | 0.426 | 0.452 | 0.079 |
| 134 | 0 | 1 | 1 | OLD | 5 | D2 | 2.5 | 2.1 | 2.5 | 40.0 | 34.0 | 40.0 | 0.0 | 88.6 | 9 | 7 | 0 | 2 | 9 | 0.206 | 0.225 | 0.225 | 0.000 | 0.062 |
| 135 | 0 | 1 | 0 | OLD | 5 | D2 | 9.8 | 8.9 | 9.8 | 199.4 | 181.4 | 199.4 | 0.0 | 167.9 | 13 | 13 | 0 | 0 | 13 | 0.072 | 0.065 | 0.065 | 0.000 | 0.000 |
| 136 | 1 | 0 | 0 | OLD | 5 | D2 | 31.0 | 26.7 | 31.0 | 852.6 | 733.0 | 852.6 | 0.0 | 995.2 | 84 | 75 | 0 | 9 | 84 | 0.102 | 0.099 | 0.099 | 0.000 | 0.025 |
| 137 | 1 | 0 | 0 | OLD | 5 | D2 | 17.5 | 15.3 | 17.5 | 403.4 | 353.8 | 403.4 | 0.0 | 506.7 | 69 | 50 | 0 | 19 | 69 | 0.141 | 0.171 | 0.171 | 0.000 | 0.103 |
| 138 | 0 | 1 | 0 | MID | 5 | D2 | 4.3 | 3.0 | 4.3 | 62.9 | 43.7 | 62.9 | 0.0 | 246.7 | 8 | 3 | 0 | 5 | 8 | 0.069 | 0.127 | 0.127 | 0.000 | 0.056 |
| 139 | 0 | 1 | 0 | OLD | 5 | D2 | 2.2 | 1.4 | 2.2 | 63.9 | 42.8 | 63.9 | 0.0 | 292.1 | 24 | 11 | 0 | 13 | 24 | 0.257 | 0.376 | 0.376 | 0.000 | 0.122 |
| 140 | 0 | 1 | 1 | MID | 5 | D2 | 10.5 | 9.3 | 10.3 | 186.7 | 164.8 | 183.5 | 97.6 | 195.2 | 31 | 11 | 13 | 7 | 18 | 0.067 | 0.098 | 0.166 | 0.365 | 0.098 |
| 141 | 1 | 0 | 1 | OLD | 5 | D2 | 4.2 | 3.0 | 4.2 | 137.2 | 97.2 | 137.2 | 0.0 | 267.1 | 17 | 14 | 0 | 3 | 17 | 0.144 | 0.124 | 0.124 | 0.000 | 0.031 |
| 142 | 1 | 1 | 0 | OLD | 5 | D2 | 8.2 | 5.9 | 8.2 | 137.8 | 99.0 | 137.8 | 0.0 | 183.2 | 18 | 9 | 0 | 9 | 18 | 0.091 | 0.131 | 0.131 | 0.000 | 0.135 |
| 491 | 1 | 1 | 1 | OLD | 5 | D2 | 7.1 | 5.9 | 7.1 | 115.7 | 96.1 | 115.7 | 0.0 | 134.6 | 10 | 6 | 0 | 4 | 10 | 0.062 | 0.086 | 0.086 | 0.000 | 0.081 |
| 561 | 1 | 1 | 0 | MID | 5 | D2 | 4.0 | 3.7 | 3.9 | 57.0 | 52.4 | 55.3 | 77.7 | 77.7 | 7 | 5 | 0 | 2 | 7 | 0.095 | 0.127 | 0.123 | 0.000 | 0.071 |
| 562 | 1 | 1 | 0 | MID | 5 | D2 | 2.2 | 1.5 | 2.2 | 30.8 | 21.5 | 30.8 | 0.0 | 155.4 | 10 | 2 | 0 | 8 | 10 | 0.093 | 0.325 | 0.325 | 0.000 | 0.141 |
| Schemes: | | | | | Totals: | | | | | | | | | | | | | | | | | | | |
| 112 | | | | | 544 | | 1244.0 | 1060.2 | 1228.8 | 29452.6 | 24694.5 | 29053.4 | 7530.8 | 32938.6 | 4226 | 2565 | 633 | 1028 | 3593 | | | | | |
| | | | | | | | | | | | | | | | | | | | Averages: | 0.104 | 0.124 | 0.143 | 0.230 | 0.086 |
| | | | | | | | | | | | | | | | | | | | Minimum (>20 accidents): | 0.050 | 0.051 | 0.051 | 0.102 | 0.056 |
| | | | | | | | | | | | | | | | | | | | Maximum (>20 accidents): | 0.381 | 0.424 | 0.602 | 0.818 | 0.420 |

## Abstract

This study of dual-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 112 new schemes, opened since 1968, on non-built-up dual-carriageway trunk roads in England. The study used the technique of generalised linear modelling 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

TRL336 *Accidents on modern rural single-carriageway trunk roads* by D A Walmsley, I Summersgill and C Binch. 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 V Kennedy, R D Hall and S Barnard. 1998 (price £50, code P)

TRL185 *Accidents at urban priority crossroads and staggered junctions* by R E Layfield, I Summersgill, R D Hall and K Chaterjee. 1996 (price £50, code P)

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