Proceedings of ROAD SAFETY AND TRAFFIC ENVIRONMENT IN EUROPE in Gothenburg, Sweden, September 26-28, 1990

- City Planning
- Speed
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Abstract (background, aims, methods, results) max 200 words:

Papers presented at the seminar were as follows: Traffic Management by Design in One Family Housing Areas (Bjoerneboe, J); Pedestrian Safety and Delay at Crossing Facilities in the United Kingdom (Hunt, J G); Safety of Cycling Children. Effect of the Street Environment (Leden, L); Analysis of Traffic Safety Regarding Public and Individual Transport (Koehler, U); Urban Traffic Network Design - A Spatial Approach (Gunnarsson, S O); Comparison of Road Safety in Different Cities (Neumann, H-J); Effects of Speed Reducing Measures in Danish Residential Areas (Engel, U); Case Study Evaluating Traffic Warning Devices with Respect to Operating Speeds and Accident Rates (Lamm, R, Choueiri, E M and Maiaender, T); Traffic Safety Effects from Traffic Calming (Brilon, W and Blanke, H); Statistical Distribution of Speeds on German Motorways (Heidemann, D); Drivers' Attitudes and Beliefs Towards Speed Limits and Speeding on Dutch Motorways (Rooijers, T).
PREFACE

The Swedish Road and Traffic Research Institute (VTI) and the Bundesanstalt für Strassenwesen (BASt), Federal Republic of Germany, were jointly organizing this international conference. The objective was to review and examine some specific road safety issues and the increasing environment problems in road traffic in different countries.

The following areas, within the field of Road Safety and Environment, were presented
- vehicles
- city planning
- speed
- vulnerable road users
- future traffic and RTI
- environment
- campaigns and publicity
- information and enforcement

Linköping October 1990

Kenneth Asp

Proceedings of ROAD SAFETY AND TRAFFIC ENVIRONMENT IN EUROPE in Gothenburg, Sweden, September 26-28, 1990:

VTI RAPPORT 362A
- Opening
- Vehicles

VTI RAPPORT 363A
- City Planning
- Speed

VTI RAPPORT 364A
- Vulnerable Road Users
- Future Traffic and RTI
- Environment

VTI RAPPORT 365A
- Campaigns and Publicity
- Information and Enforcement
CONTENTS

Program
List of participants

CITY PLANNING

Traffic Management by Design in One Family Housing Areas
Jens Bjørneboe, Norwegian Building Research Institute, Norway

Pedestrian Safety and Delay at Crossing Facilities in the United Kingdom
John G Hunt, University of Wales College of Cardiff, United Kingdom

Safety of Cycling Children - Effect of the Street Environment
Lars Leden, Technical Research Centre of Finland, Finland

Analysis of Traffic Safety Regarding Public and Individual Transport
Uwe Köhler, University of Kassel, Federal Republic of Germany

Urban Traffic Network Design - A Spatial Approach
S Olof Gunnarsson, Chalmers University of Technology, Sweden

Comparison of Road Safety in Different Cities
Hans-Jürgen Neumann, University of Transport and Communications Dresden, German Democratic Republic
SPEED

Effects of Speed Reducing Measures in Danish Residential Areas
Ulla Engel, Danish Council of Road Safety Research, Denmark

Case Study Evaluating Traffic Warning Devices with Respect to Operating Speeds and Accident Rates
Rüdiger Lamm, University of Karlsruhe, Federal Republic of Germany, Elias M Choueiri, North Country Community College, USA and Theodor Mailänder, Mailänder Ingenieur Consult, Federal Republic of Germany

Traffic Safety Effects from Traffic Calming
Werner Brilon and Harald Blanke, Ruhr University, Federal Republic of Germany

Statistical Distribution of Speeds on German Motorways
Dirk Heidemann, BASt, Federal Republic of Germany

Drivers' Attitudes and Beliefs Towards Speed Limits and Speeding on Dutch Motorways
Ton Rooijers, Traffic Research Centre, the Netherlands
ROAD SAFETY AND TRAFFIC ENVIRONMENT IN EUROPE

Gothenburg, Sweden

September 26-28, 1990

WEDNESDAY SEPTEMBER 26

OPENING

9.30 - 11.30

Chairman: Mrs Monica Sundström, Director General, Swedish Road and Traffic Research Institute (VTI), Sweden

Opening Speeches
Mrs Gunnel Färm, Deputy Minister of Transport and Communications Sweden
Prof Dr Heinrich Praxenthaler, President, Federal Highway Research Institute (BASt), Federal Republic of Germany
Mrs Monica Sundström, Director General, Swedish Road and Traffic Research Institute (VTI), Sweden

Motorization and Trends in Road Traffic
Prof Dr-Ing Karl-Heinz Lenz, Federal Highway Research Institute (BASt), Federal Republic of Germany

Traffic and Environment - What is the Problem?
Mr Börje Thunberg, Research Director, Swedish Road and Traffic Research Institute (VTI), Sweden

Traffic Safety facing Year 2000: Challenge for the Automotive Industry
Mr Jan Crister Persson, Vice President Engineering, Volvo Car Corporation, Sweden
WEDNESDAY SEPTEMBER 26

VEHICLES

13.00 - 16.30

Chairman: Prof Dr-Ing Karl-Heinz Lenz, Federal Highway Research Institute (BASt), Federal Republic of Germany

The Use of Simulation to Improve Vehicle Design
Mr François Badin, Institut National de Recherche sur les Transports et leur Sécurité (INRETS), France

Vehicle Development and Road Safety
Dr Christa Michalik, Austrian Road Safety Board, Institute of Traffic Education, Austria

The Role of the Motor Vehicle in Traffic Engineering of the Future
Dr Joachim Schmidt, Deutsche Automobilgesellschaft mbH (DAUG), Federal Republic of Germany

Automotive Crash Safety Engineering – Time for a New Approach?
Mr Hugo Mellander, Volvo Car Corporation, Sweden

The Daimler-Benz Driving Simulator – Research for Road Safety and Traffic Environment
Dipl Inf Volkhard Schill and Mr Joachim Stritzke, Daimler-Benz, Federal Republic of Germany

The VTI Driving Simulator
Prof Staffan Nordmark, Swedish Road and Traffic Research Institute (VTI), Sweden

Protection Effects of Child Restraints – Experiences from Accidents and Sled Tests with Carry-Cots
Dipl-Ing K-P Glaeser, Federal Highway Research Institute (BASt), Federal Republic of Germany
WEDNESDAY SEPTEMBER 26

CITY PLANNING

13.00 - 16.30

Chairman: Prof Niels O Jørgensen, Technical University of Denmark, Denmark

Traffic Management by Design in One Family Housing Areas
Architect Jens Bjørneboe, Norwegian Building Research Institute (Norges Byggforskningsinstitutt), Norway

Pedestrian Safety and Delay at Crossing Facilities in the United Kingdom
Dr J G Hunt, University of Wales College of Cardiff, United Kingdom

The Safety of Cycling Children. Effect of the Street Environment
Dr Lars Leden, Technical Research Centre of Finland, Finland

Analysis of Traffic Safety regarding Public and Individual Transport
Prof Dr-Ing Uwe Köhler, University of Kassel, Federal Republic of Germany

Urban Traffic Network — A Spatial Approach
Prof Dr S Olof Gunnarsson, Chalmers University of Technology, Sweden

Comparison of Road Safety in Different Cities
Dozent Dr sc techn H-J Neumann, Transport University (Hochschule für Verkehrswesen), German Democratic Republic
THURSDAY SEPTEMBER 27

SPEED
9.30 - 13.00

Chairman: Mr Gunnar Carlsson, Research Director, Swedish Road and Traffic Research Institute (VTI), Sweden

Effects of Speed Reducing Measures in Danish Residential Areas
Ms Ulla Engel, Senior Research Scientist, Danish Council of Road Safety Research, Denmark

A Case Study Evaluating Traffic Warning Devices with Respect to Operating Speeds and Accident Rates
Prof Dr-Ing Rüdiger Lamm, University of Karlsruhe, Federal Republic of Germany

Area Wide Traffic Calming Measures and Their Effects on Traffic Safety in Residential Areas
Prof Dr-Ing Werner Brilon, Ruhr-University Bochum, Federal Republic of Germany

Statistical Distribution of Speeds on German Motorways
Dr Dirk Heidemann, Federal Highway Research Institute (BASt), Federal Republic of Germany

Drivers' Attitudes and Beliefs towards Speed Limits and Speeding on Dutch Motorways
Dr Ton Rooijers, Traffic Research Centre (VSC), The Netherlands

FUTURE TRAFFIC AND ROAD TRAFFIC INFORMATICS (RTI)

(Workshop)
14.00 - 17.30

Chairman: Prof Kåre Rumar, Swedish Road and Traffic Research Institute (VTI), Sweden

Test Site West Sweden: Learning RTI and Demonstrating Its Usefulness
Mr Lars-Erik Sjöberg, National Swedish Road Administration, Sweden

Future Traffic and RTI. Status report of the Federal Republic of Germany
Dr Ing Jürgen Behrendt, Leitender Regierungsdirektor, Federal Highway Research Institute (BASt), Federal Republic of Germany

Evaluation of the Perspectives of Driving Aids based on Short Range Transmission Links between Ground and Vehicles and between Vehicles
Mr Yves David, INRETS-CRESTA, France

RTI - Current Global Projects
Mr Tage Karlsson, Director, Volvo DRIVE-SECFO, Belgium
THURSDAY SEPTEMBER 27

VULNERABLE ROAD USERS

9.30 - 13.00

Chairman: Prof Dr S Olof Gunnarsson, Chalmers University of Technology, Sweden

Riding a Moped: Acquisition of Basic Skills and Mental Effort
Dr Marcel Wierda, Traffic Research Centre (VSC), The Netherlands

An Intelligent Traffic System for Vulnerable Road Users
Mr Oliver Carsten, Senior Research Fellow, Institute for Transport Studies, United Kingdom

Traffic Related Knowledge, Attitudes and Risk Perception in Dutch Secondary School Children; Consequences for Traffic Education
Dr Jan Brinks, Traffic Research Centre (VSC), The Netherlands

Lifestyle, Leisurestyle and Traffic Behaviour of Young Drivers
Dr Horst Schulze, Federal Highway Research Institute (BASt), Federal Republic of Germany

ENVIRONMENT (WORKSHOP)

14.00 - 17.30

Chairman: Mr Göran Friberg, Director, Swedish Environmental Protection Board (SNV), Sweden

Total Environmental Impact of the Car
Mr Ulf Jansson, Volvo Car Corporation, Sweden

Environment. Status report of the Federal Republic of Germany
Dr Klaus Becker, Federal Environmental Agency (Umweltbundesamt), Federal Republic of Germany

Status report from the Netherlands
Dr M P J Pulles, Center for Energy and Environmental Studies (IVEM), The Netherlands
FRIDAY SEPTEMBER 28

CAMPAIGNS AND PUBLICITY

8.30 - 12.30

Chairman: Prof Dr Günter Kroj, Federal Highway Research Institute (BASt), Federal Republic of Germany

National Road Safety Politics – A Contradictory and Suppressed Field of Decision Making
Ms Karin Køltzow, Research Officer, Institute of Transport Economics (TØI), Norway

Motorway Driving Speed Reduction and the Associated Public Information Campaigns in the Netherlands
Dr Peter Liedekerken, Ministry of Transport and Public Works, The Netherlands

Campaigns against Drunken Driving among Young Drivers
Mr Per Studshøl, Section Eng, Danish Society of Engineers (Nordjyllands Amt), Denmark

The Effectiveness of the 1988 Police National Motorway Safety Campaign
Ms Nicola Christie, Transport and Road Research Laboratory (TRRL), United Kingdom

Improvement of Traffic Safety by Local Public Relations Campaigns
Dipl-Ing Klaus Schlabbach, Town Planning Authority Darmstadt (Bauderzernat Stadtplanungsamt), Federal Republic of Germany

A Comedy on TV to Promote Traffic Safety
Dr R D Wittink, Institute for Road Safety Research (SWOV) and Dr W J A Nelissen, Research & Marketing, The Netherlands

Road Safety as Business – Vision or Reality? The Brazilian Example
Mr J Pedro Correa, Volvo do Brasil, Brazil
FRIDAY SEPTEMBER 28

INFORMATION AND ENFORCEMENT

8.30 - 12.30

Chairman: Prof Dr Karl-Heinz Lenz, Federal Highway Research Institute (BASt), Federal Republic of Germany

A New Way of Broadcasts for Motorists
Mr Walter Melchers, Der Innenminister des Landes NRW, Federal Republic of Germany

Automatic Monitoring and Enforcement of Traffic Highway Violations
Mr Nicholas Ayland, Castle Rock Consultants, United Kingdom

Can Road Traffic Law Enforcement Permanently Reduce the Number of Accidents?
Mr Torkel Bjørnskau and Mr Rune Elvik, Research Officer, Institute of Transport Economics (TØI), Norway

A Vehicle Accident Data Recorder
Dr William Fincham, Queen Mary and Westfield College, United Kingdom

Enforcement: The Scope for Automotive Detection and Information Systems
Dr Talib Rothengatter, Traffic Research Center (VSC), The Netherlands
List of participants Road Safety and Traffic Environment in Europe September 26-28, 1990

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Schlabbach Klaus Baudezernat-Stadtplanungsamt Fed Rep of Germany
Schmidt Joachim Deutsche Automobilgesellschaft mbH Fed Rep of Germany
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Turbell Thomas VTI Sweden
Tykesson Stefan Swedish Road Adm Sweden
Vidal Pinheiro Pedro Prevencao Rodoviaria Portuguesa Portugal
Wasen Anders Newspaper Borås Tidning Sweden
Wenåll Jan VTI Sweden
Wierda Marcel Traffic Research Centre The Netherlands
Wittink Roelof SWOV The Netherlands
Yamamura Tomohiro Nissan Motor Co Ltd Japan
Åsander Sven AB Svensk Bilprovning Sweden
Örtendahl Per Anders Swedish Road Administration Sweden
Traffic Management by Design in
One Family Housing Areas

Jens Bjørneboe
Architect
Norwegian Building Research Institute
Norway
TRAFFIC MANAGEMENT BY DESIGN IN ONE FAMILY HOUSING AREAS

By architect Jens Bjørneboe, Norwegian Building Research Institute, Oslo

High quality access roads can be a problem
- Driving will be too fast
- Parents will be anxious for their children
- A large area will be tied up by road systems
- The separation of pedestrians and cars sometimes does not work
- Costs will be high

An alternative is a simple road system based on slow-moving traffic, which can result in economic savings and better outdoor areas. Roads use a varied road profile in proportion to the traffic load. Access roads are designed to the needs of children and pedestrians rather than the motorist.

The benefits of slow traffic - a field investigation
The author has done studies of thirty access roads in residential areas with one family housing. This is the predominant (60%) housing type in Norway, but little or nothing is known of traffic behavior. The aim was to find out about attitudes and behavior in order to develop better road design.

Four types of residential areas have been studied. All have about 30 houses along a road about 300 m of length, but the traffic load was dramatically different, from "no traffic" to large amounts of unwanted through traffic (ADT 20 - 5000).

Various methods have been used: automatic and handheld radar, single and group interviews with children and adults, and professional evaluation of the development. A questionnaire that was distributed to thousand households gave 791 replies.

Four recommendations for safer access roads
There is a strong desire among people to be able to drive up to their own door in one family areas (90%). Having mixed traffic in housing area is however debatable. The presumption is that the traffic is light, slow, and safe. The author suggest:

1) A "gate" to clearly indicate an area for slow traffic. The "gate" should consist of several elements like 90 degree turns, speed humps, crossing walkways, gateways, planted areas and traffic signs.

2) All access roads should be culs-de-sac with no more than 30 - 50 dwellings, an ADT less than 200 and an optimum length of about 250 m (1000 feet). The roads should be mostly one-lane, with sharp turns and strict control of visibility.

3) Playgrounds should be more attractive than the road, and as large as several single lots. Roads and paths should provide children with safe circular "round trips".

4) Speed humps should hardly be used in new developments: Radar show very different speeds from 19 to 59 kmh on the hump, depending mostly on the quality of car. Traffic don't move smoothly and silently with speed humps.
This paper discusses:
- road design in new residential areas
- conditions for a car free environment
- built-in speed management for access roads
- geometric design of culs-de-sac with light traffic
- speed management without humps

Good planning means building very different roads. Main roads should have high standard and be adapted to high speed and great traffic loads. Access roads should be designed for light and safe traffic, and speed levels of 30 km/hour or lower. They have an optimum length, of about 300 meters. (Fig.: Byggforsk 1990)

Traffic load and road design should correlate. A main road may have a capacity of about 25,000 cars per lane. Access road may typically have a daily traffic load of 50 - 100 cars per lane. The dramatic difference in traffic should be reflected in the road design. (Fig.: Byggforsk 1990)
Separation relies on discipline, from drivers and children. The Radburn development has influenced planners for over 60 years. It introduced traffic separation in one family housing areas. The architect has later complained of children playing in front of garage doors instead of using the generous green areas. Drivers may be controlled through planning. Children, however, are not easy to discipline. (Fig.: BRSCP 36/69)

Monofunctional planning doesn't always work. The principle is based on the concept of each part of the road system serving a single purpose - monofunctionalism. Separation may give pedestrians a high degree of safety, if they obey the intentions of the design. Above: Separation means that persons walking from A to B have to make a considerable detour. Traffic will be light, and it is tempting, for instance for children, to take the shortcut along the vehicle road.

Below: When traffic is less than 200 cars a day, it is more realistic to build roads for mixed use. Pedestrians must be given priority in road design and traffic rules. (Fig.: Vejdir, kbh, 1978)
Research on one family housing was needed. The author has done studies of thirty access roads in areas with one family housing. This is the predominant (60%) housing type in Norway, but little or nothing is known of traffic. The aim was to find out about behaviour and attitudes in order to improve access road design. (Fig.: Norw. off. Statistics, 1989)

Four control groups of residential areas have been studied. All have about 30 houses along a road about 300 m of length, but the traffic load was dramatically different, from "no traffic" to large amounts of unwanted through fare traffic (ADT 20 - 5000). Various methods have been used; automatic and pistol radar, single and group interviews with children and adults, and a professional evaluation of the developments. A questionnaire distributed to one thousand households gave 791 replies. (Fig.: Byggforsk 1990)
<table>
<thead>
<tr>
<th>Type of road</th>
<th>Frequency of anxiety today</th>
<th>Wants car free environment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terraced houses - car free with</td>
<td>11%</td>
<td>41%</td>
</tr>
<tr>
<td>service traffic only</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Terraced houses - &quot;car free&quot; with</td>
<td>28%</td>
<td>53%</td>
</tr>
<tr>
<td>unwanted traffic</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detached houses - culs-de-sac</td>
<td>17%</td>
<td>4%</td>
</tr>
<tr>
<td>Detached houses - through fare traffic</td>
<td>46%</td>
<td>6%</td>
</tr>
</tbody>
</table>

A car free environment is strongly wanted in terraced housing, especially where there is through traffic. Some planners advocate a car free environment for all residential areas. This contrasts strongly with the findings of our study. Parking on own site is felt to be part of the concept of the detached house. There is nothing in our material to suggest that this is dangerous or undesirable. People in culs-de-sac report the same level of anxiety as people in the best car free environment. But having mixed traffic in a housing area is debatable. The condition for mixed traffic is that it should be light, slow and safe. This can incorporated by planners when designing a development and road system.

Road design and design speed. Great expertise has been developed for designing for heavy traffic at high speed. The carriageway should be wide and almost straight. The natural speed driving on a clear road may be 80 to 110 km/hour. (Fig.: Bommen, Våre Veger, 3/89)
Safety can be corrupted into high speed. This housing area is served by a straight road, 8 meters wide. With slow driving this will give an ample measure of safety. The speed limit is 50 km/hour. The road has an ADT of less than 500, or less than a car a minute at peak traffic. Driving at the rather high design speed may be tempting. (Fig.: Byggforsk, 1990)

Access roads of a too high standard can be a problem:
- Drivers are given a high degree of freedom
- Driving will be fast on local roads
- Parents will be anxious for their children
- A large area will be tied up by road systems
- Cost will be high

Car encounters car. The width of the carriageway is of great importance for speed. When the access road is narrowed down to 3.5 m, 55 % will drive slower than 30 km/hour. (TØI, 1986)
Making roads narrow is not enough. A straight road can be taken at high speed between crossings, because of good meeting sight. According to traffic rules, those entering the road has to take care. (Fig.: Byggforsk, 1990)

The horizontal radius is important for design speed. The minimum radius is related to the square of velocity. By reducing the radius to 15 meters, design speed will be about 20 km/hour. Good access can be provided for all ordinary vehicles. (Fig.: Byggforsk, 1990)
Sharp turns are better than humps in reducing speed. To the right the diagram shows an ordinary road with speeds between 30 and 70 km/hour. A hump will bring the speed down to 25 - 30 km/hour. Some drivers, especially in good cars, traverse the hump at high speed, and everyone accelerate between humps. Traffic will be noisy and uneven.

Sharp turns and T-crossings are not seen as a provocation, and over 60% of drivers uses between 20 - 25 km/hour. Speed humps should be minimized in new developments.

Alternative access roads save money, move traffic slower and improve outdoor areas. Roads have a varied profile in proportion to traffic load. Access roads are designed to the needs of children and pedestrians rather than the motorist. Narrow, winding access roads will keep speed at a low level, even without using speed humps. (Fig.: Byggforsk, 1990)
In residential areas we should plan for slow traffic. This can be achieved by single lane roads with limited meeting sight. The perspective is gently shut by houses and vegetation, and the design speed is 20 - 30 km/hour. (Fig.: Vejdir., Kbh, 1978)

Everyone will calm down here. This rural road is the entrance to an upper class residential area. Landscaping and beautiful old trees dominate the road, and design speed is probably around 20 km/hour. (Fig.: Byggforsk, 1990)
Design speed can be set at any level. The hard surface may be slabs to emphasize landscaping and vegetation. In this area drivers will be motivated to drive at walking speed, and perhaps park the car outside the housing group. (Fig.: Byggforsk, 1990)

Not more than 200 cars a day. Access roads should be culs-de-sac with no more than 30 - 50 dwellings, an ADT less than 200 which means about 5 cars per 15 mins at most. The length is about 300 meters (1,000 feet). The roads should be predominantly single lane, with sharp turns and strict control of visibility. (Fig.: Performance Streets, Pennsylvania, 1980)
A "gate" should clearly indicate an area for slow traffic. It should consist of several elements, a gateway, 90 degree turns, speed hump, crossing walkway, planted areas and traffic signs. Speed has to be forced down to a level of about 20 km/hour, when a 90 degree turn is followed by a hump. Visual and formal measures can underline the transition from collector to access roads. (Fig.: Byggforsk, 1990)

Speed management can be simple. This small development has a road system that inspires slow driving. Short straight sections, and sharp turns will give a moderate speed level. (Fig.: Byggforsk, 1990)
A successful development. A beautiful woodland site with narrow access roads gently following the contour lines. The collector road is short and ends in culs-de-sac with an optimum length of 300 meters.

Playgrounds should be more attractive than the road, and as large as several single lots. The green common area is placed in the middle of a group of homes. This area immediately facing front doors will be carefully kept by the residents. Speed will be reduced to an acceptable level by means of several sharp turns, without "speed reducing means". Roads and paths should provide children with safe circular "round trips" without entering collector roads.
Conditions for mixed traffic:

* slow speed level
  85 percentile below 20 - 25 km hour

* light traffic load
  ADT less than 200 cars a day at most
  or no more than 5 cars per 15 mins at most

* safety measures
  very important for children

Recommended solutions for mixed traffic:

* slow speed level:
  mostly one lane, carriageway width of 3.5 m
  wide shoulders
  sharp turns, full 90 degrees
  humps hardly necessary

* light traffic load:
  culs-de-sac with less than 50 houses
  and length less than 300 m

* safety measures:
  safe inner circle for the children
  large, really attractive playgrounds
  traffic gate between access and collector roads
  control of sight-lines

Summing up
To have slow, light and safe traffic in access roads is to large extent a matter of design. The design speed is dependent on the geometry of the road, which barely should fit the larger vehicles. The amount of traffic will be mostly dependent on the number of homes in a cul-de-sac. Safety measures are features of a good development plan.
Back to Scaf. In Norway it is almost forgotten that Scaf in fact recommended mixed traffic and shared spaces in one family housing. The author finds this a good principle for new developments. (Fig. Scaf 1967)

LITTERATURE


Pedestrian Safety and Delay at Crossing Facilities in the United Kingdom

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United Kingdom
Proposed paper by: Dr. J.G. Hunt
University of Wales College of Cardiff, Cardiff, United Kingdom.

Title: Pedestrian safety and delay at crossing facilities in the United Kingdom.

ABSTRACT

This paper is based on the results of comprehensive studies of pedestrian delay and safety at a range of crossing locations in the United Kingdom over a period of ten years from 1979. In the United Kingdom, where there is no legal requirement preventing a pedestrian crossing the carriageway at any location, Zebra and Pelican crossings are installed to assist pedestrians at mid block. At junctions a pedestrian stage may be provided on one or more arms provided the traffic capacity at the junction is not exceeded.

The Zebra crossing, which is uncontrolled and relies on drivers yielding right of way to pedestrians on the carriageways, is shown to function satisfactorily at low flow. However, at high vehicle flow pedestrians may have difficulty establishing right of way while high pedestrian flow may dominate the carriageway thus inducing high vehicle delay. Pelican crossings, which were introduced to overcome some of the disadvantages of the Zebra, combine the flexibility of the Zebra with the protection afforded to pedestrians by the red light at traffic signals. Pelican crossings may operate on a fixed time or vehicle actuated basis with a pedestrian precedence period granted following pedestrian demand. On wide roads dual Pelicans with a staggered layout are employed. More recently the operation of Pelicans within UTC systems has limited the periods during which pedestrian precedence periods can be initiated. The paper reviews the performance of Zebra and Pelican crossings based on data collected at over 100 locations in the United Kingdom and an accompanying analysis of accident statistics. It is shown that pedestrian delay can increase significantly when Pelicans are linked within UTC systems and that pedestrian delay is usually lower at random crossing locations than at sites where specific facilities are provided. Studies of pedestrian-vehicle interaction at junctions demonstrated that the provision of a pedestrian stage on one arm may be to the disbenefit to pedestrians crossing on other arms.

More recent developments including the use of speed humps or reduced carriageways width at crossing locations, and facilities based on pedestrian detection using infra red or pressure sensitive pads are briefly described.
PEDESTRIAN SAFETY AND DELAY AT CROSSING FACILITIES IN THE UNITED KINGDOM

J.G. Hunt
Lecturer
School of Engineering, University of Wales College of Cardiff, United Kingdom.

1. INTRODUCTION

Most journeys involve use of the highway as a pedestrian with one third of all travel time spent walking. While pedestrians are more adaptable than motor vehicles they are also vulnerable in any possible interaction. Interaction is most likely to occur when pedestrians have to cross the carriageway in order to continue their journey and it is not surprising that more than 70% of pedestrian fatalities occur while pedestrians are crossing the road. In most countries, except at formal crossing locations or within designated pedestrian areas, vehicles have priority on the carriageway and pedestrians wishing to cross the road must either wait for gaps in the traffic stream or walk to the nearest formal crossing location either at a junction, possibly as part of signal control, or at mid-block. In the United Kingdom there is no legal requirement preventing a pedestrian crossing the carriageway at any location at any time so that a pedestrian may, for example, cross despite an adverse indication at pedestrian signals.

While the United Kingdom has one of the best records in the EC, and possibly the world, in terms of fatal accidents per capita or per motor vehicle, the number of pedestrians killed per 100,000 of population and the percentage of fatal accidents involving pedestrians in the United Kingdom is high in relation to many other countries. More than 1700 pedestrians die each year on roads in the United Kingdom and it is in order to reduce these appalling statistics that the Department of Transport has launched a series of initiatives to improve pedestrian safety. However the extent of the success of these initiatives will depend on resources and public co-operation. It is interesting to speculate on the comparative level of priority which would accompany 1700 United Kingdom fatalities in aviation accidents in one year!

The elimination of accidents to pedestrians who are crossing the road would require the complete segregation of pedestrians and vehicles by the use of guide separated crossings such as footbridges or subways and a legal requirement for their use. Such an extreme solution is clearly not feasible in terms of either cost or environmental impact; it would also be extremely difficult to achieve public co-operation for such a restrictive policy. Currently in the United Kingdom, as in other countries a mix of facilities is provided dependent on site and traffic conditions and on pedestrian demand. Between junctions and at non signalised junctions, refuges, Zebra or Pelican crossings are installed to assist pedestrians. At junctions a pedestrian
stage may be provided on one or more arms provided the traffic capacity of the junction is not exceeded. The provision of pedestrian crossing facilities is at the discretion of the Local Highway Authority who normally act on the basis of guidelines issued by the Department of Transport. At a time when traffic congestion in cities is again becoming a major issue there is some concern that in seeking to maximise the traffic capacity of existing roads and junctions, pedestrians will be disadvantaged in terms of delay and possibly safety. Such concern may be increased by the current policy of taking traffic control and management decisions on the basis primarily of costs associated with alternative options. With pedestrian time at £4.24 per pedestrian hour and vehicle time at £5.60 per hour, a substantial concentration of pedestrian activity would be required to enhance existing provision. This paper considers pedestrian delay and safety based on observations at more than 100 locations in the United Kingdom.

2. CROSSING BETWEEN JUNCTIONS

2.1 Random roadside locations

Pedestrians will usually seek to minimise their journey time and distance by crossing the road at the nearest convenient location. Pedestrian journeys are both diverse and individual and the majority of crossing movements are distributed widely over the road network with concentrations of pedestrian crossing movements restricted to relatively few locations mostly in town centres. Where vehicle flow is low, as is the case in residential streets, pedestrians experience little difficulty in crossing the road and will usually minimise any delay by simply taking advantage of any crossing opportunity when it arises, i.e. pedestrians continue their journeys while scanning the traffic stream and do not necessarily wait at the kerbside. As traffic flow increases pedestrians experience more difficulties in selecting a suitable crossing opportunity and may be delayed at kerbside while waiting to continue their journey. Figures 1 and 2 show the variation in pedestrian mean delay and percentage of pedestrians delayed with vehicle flow while crossing the road at random points. The data shown are derived from a simulation model developed on the basis of observations at 42 sites in 6 regions of the United Kingdom. Available evidence suggests that the majority of pedestrians experience little difficulty in crossing the road when the combined vehicle flow is below 1000 veh/h. However the averaging process used in calculating pedestrian mean delay conceals substantial delays experienced by a minority of pedestrians as demonstrated by Figure 3 which shows the distribution of pedestrian delay for one of the regions at which observations were made.

As vehicle flow increases so pedestrians experience more difficulty in crossing the road with a significant proportion of pedestrians prepared to adopt more sophisticated patterns of
movement to minimise their delay. Observation indicated that the sequential distribution of vehicle inter arrival times also had a significant impact on the level of difficulty experience by pedestrians crossing the road. The formation of platoons within the traffic stream, as for example downstream of signal controlled junctions, can provide clear crossing opportunities during periods of zero or low traffic flow. Under these conditions the provision of pedestrian refuges is especially beneficial as indicated by Figure 1 where pedestrian mean delay at refuges may be compared with comparable results for a random crossing point.

2.2 Zebra crossings

Zebra crossings are uncontrolled, and successful operation depends on drivers yielding right of way to pedestrians on the carriageway. Effectively an implicit priority to vehicles changes to priority to pedestrians for those pedestrians who are prepared to step onto the carriageway. The pedestrian priority continues for the duration of pedestrian occupation of the crossing although in practice vehicle drivers move off as soon as the appropriate section of the carriageway is clear of crossing pedestrians. Under conditions of low vehicle and pedestrian flow the effect is to reduce pedestrian waiting times at a cost of a marginal increase in vehicle delay. Vehicle delays are kept to a minimum as vehicle drivers are required to stop only while there are pedestrians on the carriageway. The knowledge that vehicle drivers are required to yield right of way results in pedestrians accepting gaps in nearside traffic which may be 1-2 seconds less than would otherwise be the case with a consequent decrease in pedestrian mean delay as shown by Figure 1. Further, the pedestrian crossing task is simplified as they have only to consider traffic on the nearside. Under ideal conditions the Zebra crossing offers a simple transfer of periods of precedence from vehicles to pedestrians with the duration and frequency of the precedence dependent on pedestrian demand. However at high vehicle flow pedestrians may have difficulty in establishing right of way while high pedestrian flow may dominate the crossing with a consequent unacceptable increase in vehicle delay as shown in Figure 5. In urban areas where road space utilisation is high the possibility of severe traffic congestion resulting from high pedestrian flow using a Zebra crossing is considered unacceptable from a traffic management viewpoint so that under these conditions new Zebras are not installed and existing Zebras are replaced with Pelican crossings.

2.3 Pelican crossings

Pelican crossings were introduced to overcome some of the disadvantages of the Zebra while attempting to retain the flexibility of allowing vehicles to depart once the crossing is clear of pedestrians. Pedestrians are afforded the protection of a red light to vehicles while establishing their presence on
the crossing. The basic Pelican signal sequence is shown in Figure 4 where the range of permissible timings for each stage is also shown. Pelican crossings may operate on a fixed time or vehicle actuated basis with a pedestrian precedence period granted, subject to the expiry of a specified vehicle precedence period, following pedestrian demand registered by operating a push button. The 'Green Man' signal is intended to allow pedestrians to establish priority on the crossing with the 'Flashing Green Man' allowing pedestrians already on the carriageway to retain precedence to enable them to reach the far side of the road. The duration of the 'Green Man' and 'Flashing Green Man' aspects are determined primarily on the basis of road width. Although Pelican crossings have now been in use in the UK for more than 20 years, many pedestrians still do not understand the purposes of the 'Green Man', 'Flashing Green Man' sequence with both Local Highway Authorities and the Department of Transport receiving complaints from members of the public that the pedestrian precedence period allows insufficient time for pedestrians to cross the road in safety and without undue harassment from waiting vehicles. In response to comments received the guidance issued by Department of Transport in 1987[3,4] allowed the use of an extended 'Green Man' or vehicle red thus improving conditions for pedestrians. However at the same time the Department of Transport also allowed a potential disbenefit to pedestrians by specifying that Local Highway Authorities could extend the vehicle precedence period to 60 seconds if so required by site conditions.

At Fixed Time Pelican crossings operating independently the time allocation between pedestrians and vehicles, under conditions of heavy pedestrian demand, is determined by the length of vehicle precedence period. Local Highway Authorities, in order in minimise traffic congestion, will usually select a vehicle precedence period to ensure adequate vehicle capacity at peak periods. The controllers on Fixed Cycle Pelicans do not allow variation in the preset vehicle precedence period so that the limit required for peak period operation will also apply during the off peak period. Figures 5 and 6 show the variation in vehicle and pedestrian mean delay with vehicle flow for a range of vehicle precedence periods. For comparative purposes the pedestrian mean delay and percentage of pedestrians delayed at a Fixed Time Pelican with a vehicle precedence of 30 seconds is shown in Figures 1 and 2. It is clear that pedestrian mean delay at a Fixed Time Pelican usually exceed the delay at a Zebra or random crossing location except where the vehicle flow is high.

It should be noted that the pedestrian mean delay at Pelican crossings reflects the combined effect for those pedestrians who act in accordance with the signal sequence together with those pedestrians who cross during the Red Man stage. Figure 7 shows the percentage of pedestrians crossing during the Red Man stage corresponding to the pedestrian mean delay data shown in Figure 1. Particularly at low vehicle flow combined with a high vehicle precedence period, it is clear that the pedestrian
precedence period provides only for those pedestrians who are unwilling or unable to cross in gaps in the traffic. It may be postulated that gap crossing at Pelican crossings is potentially more hazardous than at random points along the road as drivers approaching a green signal may not expect to see pedestrians on the carriageway.

Vehicle actuated Pelicans were introduced initially for use on higher speed (85 percentile speed greater than 50 km/h and less than 85 km/h) roads where vehicle drivers had difficulty in stopping at the expiry of the vehicle precedence period. At Vehicle Actuated Pelicans, following pedestrian demand, a pedestrian precedence period may be initiated during a gap in vehicle flow if this occurs before the expiry of the vehicle maximum precedence period. The use of Vehicle Actuated Pelicans was later extended to urban areas where a speed limit of 50 km/h is in force. Vehicle and pedestrian mean delay at a typical Vehicle Actuated Pelican crossing are shown in Figures 5 and 1 respectively with the percentage of pedestrians crossing during the Red Man stage shown in Figure 7. Since vehicle actuation allows a change of precedence to follow a gap in vehicle flow a reduction in the percentage of pedestrians crossing during the Red Man period may be expected. However in practice pedestrians are able to cross the road in gaps which are insufficient to cause a change to pedestrian precedence and will also usually act in anticipation of such a change.

On roads more that 15 m wide a dual Pelican with a staggered layout is usually provided. Where the two Pelicans are unlinked substantial pedestrian delay, approaching twice the level on a single Pelican, will usually result. Linking is usually only possible for one direction of pedestrian flow and would probably cause a disbenefit to pedestrians travelling the non favoured direction.

In recent years UTC systems such as TRANSYT and SCOOT have been installed in most UK cities. The main objectives of UTC systems are to reduce vehicle delay and optimise capacity within an area by co-ordinating the operation of traffic signals based either on a series of fixed time plans as with TRANSYT or on a vehicle responsive basis as with SCOOT. The effective operation of such systems depends on the ability to predict the arrival times of platoons of vehicles at all signals within the area. Demand dependent facilities such as roundabouts or Zebra crossings may disrupt the planned progression and are replaced by more suitable alternatives. Pelican crossings within UTC areas are normally operated on a fixed cycle basis and constrained so that a pedestrian precedence period is only allowed to occur once during each UTC cycle. Although alternative strategies in which Pelicans are allowed to double cycle, may be employed the overall effect is to substantially increase both pedestrian mean delay and the percentage of pedestrians crossing during the Red Man stage as demonstrated in Figures 2 and 7 respectively. The increase in the percentage of pedestrians crossing during the 'Red Man' stage results from the increased waiting time for pedestrians
and from the increased opportunities available with platooned vehicle arrivals associated with traffic movement in a UTC system.

2.4 Safety

It is clear from the preceding sections that pedestrian delay is usually lower at random crossing locations than at sites where pedestrian crossing facilities such as Pelicans are provided. At Pelican crossings in areas operating UTC and at Pelicans with vehicle precedence periods exceeding 30 seconds, pedestrian delays may exceed the 30 second limit at which it is considered that pedestrians become increasingly impatient[11]. Although any relationship between delay and risk taking remains unproven, observations of pedestrian behaviour indicate that some pedestrians are prepared to accept smaller gaps in traffic flow as waiting time increases. However a before and after study of accident frequency at pedestrian crossings in areas where UTC has been installed found no significant change in pedestrian accidents associated with increased pedestrian delay[12].

Accident statistics for the UK for 1984[1] indicate that 14% of pedestrian fatalities occurred while crossing the road on or within 50m of a pedestrian crossing and 60% occurred while crossing the road elsewhere. Although the UK record for fatalities on or near a pedestrian crossing is good by international standards, the data in relation to any reasonable estimate of relative crossing movements at different locations do give rise to some concern that the risk to pedestrians is higher within a 100m section of road containing a pedestrian crossing than elsewhere on the road network. A number of explanations are available of which the most plausible is that pedestrian crossings are generally only installed at locations with a high concentration of pedestrian crossing movements or where there was previously a high rate of injury accidents involving pedestrians.

Public perception is that Pelican crossings are safer than Zebra crossings; where pedestrians believe a road crossing location is dangerous, or at locations where there have been a number of accidents, public demand is for the installation of a Pelican. Pelican crossings are also considered to offer particular benefits to the young, the elderly and disadvantaged groups. However accident studies have indicated that, given similar site conditions there is little difference in pedestrian accident rates for Pelican or Zebra crossings[4].

3. CROSSING AT SIGNAL CONTROLLED JUNCTIONS

The objective of assisting pedestrians to cross the road safely and without excessive delay at signal controlled junctions is constrained by the requirement to ensure sufficient capacity and minimum delay for vehicles. At junctions, where two or more roads intersect it is often difficult, particularly at
peak periods, to provide adequate vehicle capacity; the inclusion of an all red period necessary for a separate pedestrian stage may not be acceptable in terms of additional vehicle delay and congestion which could result. A more acceptable arrangement is to use an alternative phasing arrangement which allows a protected signalled crossing to be provided for pedestrians on one or more arms without a significant reduction in vehicle capacity.

The sequence and timing of a pedestrian display at signal controlled junctions are specified by the Department of Transport[5] and are similar to those specified for Pelican crossings. However at signal controlled junctions the Green Man to pedestrians is followed by a black out (no signal) to pedestrians and a red light to vehicles, replacing the Flashing Green Man to pedestrians, Flashing Amber to vehicles at the Pelican.

The effect of the inclusion of a separate pedestrian stage on optimum cycle time, vehicle and pedestrian delays has been demonstrated by Hunt and Khalil[10] using OSCADY and simulation. Where pedestrians were assumed to cross only during the Green Man stage, pedestrian mean delay exceeded 30 seconds for vehicle flows above 600 veh/h. Where no separate pedestrian stage was provided and pedestrians were allowed to gap cross pedestrian mean delay remained below 20 seconds for vehicle flows below 1200 veh/h. The lower pedestrian mean delay in the absence of a separate pedestrian stage was attributed to the shorter cycle time and lower degree of saturation when an all red to vehicles was not shown.

The attention given to pedestrian requirements at signal controlled junctions and the success of any measures implemented will usually be judged at least in part on the basis of accident statistics. Table 1 shows that for the period 1980 to 1984 in the UK there were more pedestrian casualties at junctions with traffic signals than at either Zebra or Pelican crossings. Assessment of relative risk is difficult since it must take account of many factors including pedestrian/vehicle conflict at each location for which data are not available. Hall[13] in a study of personal injury accidents occurring during the four years 1979-82 at a sample of 177 four arm traffic signal junctions reported that pedestrians were involved in 35% of all fatal and serious accidents. Hall also reported that for total pedestrian accidents the presence of a pedestrian stage indicated slightly higher pedestrian accident rates although its significance was somewhat marginal. Overall the data justify continuing concern about pedestrian safety when crossing the road at signal controlled junctions.
Table 1 Pedestrian casualties in the UK for the period 1980-84

<table>
<thead>
<tr>
<th>Location</th>
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<th>Casualties</th>
<th>Estimated casualties per location</th>
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<tr>
<td>Pelican</td>
<td>8,000</td>
<td>609</td>
<td>22,537</td>
</tr>
<tr>
<td>Junction with traffic lights</td>
<td>7,000</td>
<td>601</td>
<td>28,449</td>
</tr>
</tbody>
</table>

4. FUTURE PROVISION

In a situation where road space is shared between different types of user and traffic congestion is increasing, provision to enhance pedestrian safety and convenience will only be possible if sufficient priority and resources are given to this vulnerable group of road users. The provision of additional facilities for pedestrians or the allocation of additional precedence periods at existing facilities may have an adverse effect on vehicle delay and highway capacity.

It is possible that modern technology will allow even more efficient use to be made of existing road space. Currently the Transport and Road Research Laboratory, as part of an EEC Drive Project, are evaluating an alternative arrangement for a pedestrian signalling system at junctions. The proposed arrangement involves the automatic detection of pedestrians to control pedestrian signal timings and the relocation of the pedestrian Red/Green man aspects to the near side of the road so that pedestrians, once on the crossing, will not be confused by changes in the pedestrian signal aspect. If successful the arrangement could also be used at midblock crossings so that pedestrian signal facilities would be similar at all crossing sites.

Pedestrians also benefit from increasing implementation of approaches developed in the urban safety project[14]. The urban safety project evaluated the effect of area appraisal and treatment on road safety in five urban areas in the UK. The techniques used were similar to those associated with traffic calming in Germany[15], environmentally adapted roads in Denmark[16] and special areas termed 'woonerven' in the Netherlands[1]. Essentially the techniques involve giving greater precedence, within designated areas, to pedestrians and cyclists by the use of measures such as road humps, carriageway narrowing and the move extensive provision of central refuges. Interim results indicate that the new approach has been successful in reducing accidents with many UK authorities bringing safety into a more central role in their new engineering policies. It is essential that this policy is continued and expanded and that pressures to give higher
priority to vehicle traffic to relieve congestion, at the expense of provision of pedestrian facilities, are resisted.

5. REFERENCES


Fig. 1. The effect of crossing type on pedestrian mean delay
Fig. 2. The effect of crossing type on percentage of pedestrians delayed.
Fig. 3. The distribution of delay for delayed pedestrians

![Graph showing the distribution of delay for delayed pedestrians]

**VEHICLE ASPECT**

<table>
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<tr>
<th>GREEN</th>
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<table>
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<td>F</td>
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<td>C</td>
<td>D</td>
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</table>

**PEDESTRIAN ASPECT**

Fig. 4. The Pelican signal cycle
Fig. 5. The effect of crossing type on vehicle mean delay
Fig. 6. The effect of vehicle precedence period on pedestrian mean delay at Pelican crossings
Fig. 7. The effect of Pelican operating strategy on pedestrians crossing during the 'RED MAN' at Pelican crossings.
The Safety of Cycling Children
Effect of the Street Environment

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THE SAFETY OF CYCLING CHILDREN. Effect of the street environment

A field survey of about 14 000 schoolchildren between the ages of 6 and 16 has been conducted in five Swedish, one Norwegian and three Finnish towns. The schoolchildren described the road accidents they had been involved in over the past year. In addition, they drew on a map the route they had cycled the previous day and described places they thought were particularly dangerous in their neighbourhood. In the areas there were a total of slightly more than two thousand intersections and more than three thousand stretches of road. These have been inventoried and the number of schoolchildren cycling at each place has been calculated. The towns have lent assistance by providing details of the volume of motor traffic.

The starting point for the project was to develop the model for classifying the safety standards of pedestrian and bicycle routes in residential neighbourhoods as described in TRÄD. TRÄD’s description of the safety standard of a P/C path which crosses minor connecting roads seems to be less than satisfactory. On the whole, though, TRÄD appears to have achieved its objective. An alternative model, SESAM is suggested. SESAM stands for the Swedish terms corresponding to interaction, no steep gradients, special phase for signal-controlled bicycle crossings, separate motor traffic from P/C traffic and reduce motor vehicle speed. Interaction can be improved by, for example, locating bicycle crossings at an intersection immediately adjacent to the intersecting road, reducing the number of directions from which motor traffic arrives at an intersection, such as by limiting the number of approaches and exits or by prohibiting left turns, routing bicycle paths to cross streets with motor traffic on stretches of road instead of at intersections and improving sight conditions.
THE SAFETY OF CYCLING CHILDREN. EFFECT OF THE STREET ENVIRONMENT
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1. INTRODUCTION

The study, which is described in detail by Leden (1989), was based on surveys carried out in some 40 schools at seven localities in Sweden, three in Finland and one in Norway between 1983 and 1987, when a total of about 14 000 schoolchildren in classes 1 - 9 supplied details about where they cycled, the traffic accidents in which they had been involved, and the locations they thought were dangerous. The study was also based on an inventory of about 5 000 locations in the central parts of the districts and the neighborhoods where the children lived and on details of the motor-traffic flows.

Write down here why you think the place is dangerous (draw a diagram if this will make it clearer)

Fig. 1. A 5th form pupil thinks it dangerous to use the pedestrian crossing at the intersection because "You feel all dizzy".
An example of a dangerous intersection is shown in Fig. 1. Imagine you are 12 years old what will you find dangerous at the intersection? A 5th form pupil thinks it dangerous to use the pedestrian crossing at the intersection because "You feel all dizzy". Other common reasons for considering this type of intersection dangerous were "Vehicles arrive from several directions", "Turning motor traffic" and "Sight obstructed". These explanations proved to be essential.

Data about dangerous places was aggregated. The most common reasons for considering intersections and stretches of road dangerous were "Sight obstructed", "Intensive motor traffic" and "Motorists drive fast".

2. METHODOLOGY

To find a method of estimating the bicycle-traffic flow was very important. School children drew where they cycled the previous day on a map. The questionnaire was preferably not to be answered if it had rained a lot the day before, since it is known from experience that bicycle traffic is sensitive to weather conditions. Bicycle-traffic flow has been estimated by compiling statistics of the number of schoolchildren passing each individual intersection and stretch of road.

Fig. 2 is a summary showing how schoolchildren cycled in Esbovik. Two different surveys were carried out one in 1984 and the other in 1987. The band width corresponds to the number of cycling schoolchildren. A white area indicates a decrease and a black area an increase in flow between the two surveys.

I also needed a method to estimate motor-vehicle traffic. Motor-vehicle traffic flows were based on the traffic censuses carried out by the municipalities. The flow was divided into nine classes and covers all intersections and all stretches of road in the studied areas. In regard to intersections and stretches of road where no statistics are available, the flow has been estimated on the basis of comparable figures.

It is natural to calculate the risk of collision as the number of collisions between cycling schoolchildren and motor vehicles per year divided by the number of cycling children entering the intersection annually. Such a simple risk measure was proved to be better than more complicated ones.

In an analogous way, the number of collisions with motor vehicles per hundred thousand km cycled by schoolchildren has been used as a risk measure for stretches of road.
3. RESULTS

Let us look at an example of the results (Fig. 3). The collision risk is calculated as the number of collisions between cycling schoolchildren and motor vehicles divided by the number of cycling children entering the intersection. The risk of collision when cycling on the carriageway (regardless of whether the intersection is signal-controlled or not) is not affected by the total motor-traffic flow entering the intersection. This result is very interesting and support the theory of risk homeostasis (Wilde, 1982). According to this theory the accident rate is ultimately dependent on one factor only, the target level of risk in the population concerned which acts as the reference variable in a homeostatic process relating accident rate to human motivation. I will discuss this further below.
On the other hand, the risk of collision when cycling on a cycle-crossing appears to increase with the total volume of motor traffic entering the intersection. However further analyses showed this not to be true. The figure indicates that in average the risk of collision for cyclists is comparatively high in intersections with cycle-crossings. This is true especially if the intersection is signal controlled. I will explain later why.

3.1 Model for describing how street design and control influence the collision risk

I developed a model for describing how street design and control influence collision risk. In this connection, the first and most important starting-point was to design the model with the primary aim of reducing high accident risks where the types of accident having the most serious consequences are concerned.

A survey of the average number of days of hospitalization for different modes of transport showed that the periods of medical care were longest on the average for accidents involving a cycling child and a motor vehicle, followed by accidents involving a child on foot and a motor vehicle, see Fig. 4.
Studies of fatal accidents reported by the police in Sweden showed that most of the children killed in 1986 in traffic were to be found among those cycling, on foot and riding as passengers in a car, in that order. Against this background, I decided to base the model chiefly on the risk of collision between cycling children and motor vehicles. This was a very important decision.

I defined three different standard levels:
- Green standard for a low collision risk.
- Yellow standard for a moderate collision risk.
- Red standard for a high collision risk.

How intersections shall be classified will first be explained. The criteria are to be read in the order they appear. The first criterion to coincide with the actual conditions determines the standard.

Criteria for a red standard:
- speed level of motor traffic 60 kph or more,
- traffic signals showing bicycle traffic on a cycle crossing a green light at about the same time as turning motor traffic,
- existence of a steep gradient (a gradient in excess of about three per cent) at a cross-road,
- a cycle-crossing moved back 3 - 15 m from the main road. (Different possibilities in locating cycle-crossings is explained in Fig. 5.)
Fig. 5. Different possibilities in locating cycle-crossings on a crossing road.

Criteria for a green standard:
- speed level of motor traffic 30 kph or less,
- traffic signals with a special phase for cyclists,
- P/C crossings.

If none of the above criteria coincide with actual conditions, the intersection has a yellow standard.

Green standard will correspond to a collision risk of at most 0.25 collisions with a motor vehicle per hundred thousand cycling children, a yellow standard 0.25 - 0.75 and a red standard more than 0.75.

For stretches of road the model is simple. Separate cycle tracks and stretches of road with pedestrian and cycle-paths have a green standard. Other stretches of road have a yellow standard.

The following reasoning explains why stretches of road without a cycle-path have a yellow standard and not a red standard. On the average, stretches of road without a cycle-path were 120 m long and stretches of road with a cycle-path 160 m long. For the sake of argument, let us assume that there are six intersections for every km. The collision risk on stretches of road without a cycle-path averages out at 1.8 collisions per hundred thousand km cycled by the children or, spread out over six intersections, 0.3 (1.8/6) collisions per hundred thousand cycling children at each intersection. This falls within the interval for a yellow standard at intersections having 0.25 - 0.75
collisions per hundred thousand cycling children.

In conclusion, a verbal description summarizing the results is given:

- **Segregate** motor traffic from pedestrian and cycle traffic. Motor traffic is best separated from P/C traffic by constructing completely separate P/C paths or P/C streets with grade-separated crossings with the motor traffic network.

- **Improve interaction.**

*Interaction*, for example, can be improved by:

- reducing the *speed* of motor traffic and of bicycle traffic. (Road humps reduce motor traffic speed efficiently. Steep gradients are dangerous because bicycle traffic speed becomes high.)

- locating cycle-crossings at an intersection immediately next to the main road.

- allowing cycle-paths to cross streets with motor traffic at a *longer distance* from intersections,

- reducing the *number of directions* from which motor traffic enters an intersection, for example by limiting the number of approaches and exits or by prohibiting certain directions of travel, see Fig. 6,

- avoiding traffic signals showing bicycle traffic on a cycle-crossing a green light at the *same time* as turning motor traffic.

- improving *visibility* conditions.

### 3.2 Is it worthwhile building cycle-paths?

Is it worthwhile building cycle-paths? I shall throw light on this question from the viewpoint of road safety with an example based on the results of this study. The collision risk on stretches of road without a cycle-path is on the average about 1.8 collisions involving a motor vehicle per hundred thousand km cycled by the schoolchildren. On stretches of road with a cycle-path it averages about 0.2, that is to say the collision risk *diminishes* by an average of about 1.6 collisions per hundred thousand km cycled when a cycle-path is built. There are about six intersections in every km, that is to say on the plus side there will be about 0.3 (1.6/6) collisions per hundred thousand cycling children at each intersection.
Fig. 6. Improve interaction by reducing the number of directions from which motor traffic enters an intersection, for example by allowing cycle-paths to cross streets with motor traffic at a longer distance from intersections, by limiting the number of approaches and exits or by prohibiting certain directions of travel.

The average collision risk when children cycle through an intersection on the carriageway is about 0.5 collisions involving a motor vehicle per hundred thousand cycling children. To avoid an increase in the total collision risk if a cycle-path is built, the intersections must be designed so that the collision risk is at most 0.8 collisions per hundred thousand cycling children.

Consequently, it will not pay to build cycle-paths if:

- the cycle-crossings at intersections are moved back 3 - 15 m from the main road (collision risk approx. 1.3 collisions with motor vehicles per hundred thousand cycling children), see Fig. 7,

- or if intersections controlled by traffic signals are arranged so that turning motor traffic is shown a green light at about the same time as cyclists on the parallel cycle-crossing (collision risk approx. 2.8 collisions with motor vehicles per hundred thousand cycling children).
On the other hand, it will pay to build cycle-paths:

- if the cycle-paths end before the intersections (collision risk approx. 0.5 collisions with motor vehicles per hundred thousand cycling children at uncontrolled intersections and approx. 0.3 at intersections controlled by traffic signals),

- if the cycle-crossings are located immediately next to the main road (collision risk approx. 0.4 collisions with motor vehicles per hundred thousand cycling children),

- if cyclists are given a special phase on cycle-crossings controlled by traffic signals (collision risk nearly 0 collisions with motor vehicles per hundred thousand cycling children),

- and if the cycle-paths cross streets with motor traffic at a longer distance from intersections (collision risk 0.1 collisions with motor vehicles per hundred thousand cycling children) or are grade-separated (collision risk nearly 0 collisions with motor vehicles per hundred thousand cycling children).

Fig. 7. It will not pay to build cycle-paths if the cycle-crossings at intersections are moved back 3 - 15 m from the main road. There will be almost no interaction between cyclists and motor vehicles and the risk of collision will be high.
4. DISCUSSION

The present study deals mainly with the safety of children cycling in traffic, that is to say children age 6 - 16 years. Although the group is not homogeneous, it has been treated as a single unit all the same. Neither has anything emerged to speak against such a procedure. Incidentally, the problems are in many respects the same for children as for adults.

A good example of this is the danger of two-way bicycle traffic at an intersection controlled by traffic signals where cyclists and turning motor traffic are shown a green light at about the same time, which has earlier been found to result in an unacceptably high accident risk for cyclists in general (Linderholm, 1984).

Another example of this is the danger of locating bicycle crossings some distance away from the main road. According to Lauwers (1988) cycle paths at intersections in built-up areas should be brought close to the main road before intersections in order to increase visibility and heighten awareness.

According to Wilde (1982) it follows from the Theory of Risk Homeostasis that lasting accident reduction (per time unit of exposure or per capita) cannot be achieved by means of merely providing road users with more opportunity to be safe, but that safety can be enhanced by measures that increase people's desire to be safe. There are three types of motivational countermeasures to improve traffic safety: (1) those which discourage driving, while encouraging the use of safer means of transportation; (2) those which discourage specific unsafe driving acts; and (3) those which increase the costs of accidents to individuals involved and increase the benefits of not having an accident.

As mentioned above the risk of collision when cycling on the carriageway is not affected by the total motor-traffic flow entering the intersection. This result support the theory of risk homeostasis (Wilde, 1982). Note that this example deals with an environment with a comparatively low level of risk.

According to my result collision risk is influenced by street design and control. Better interaction gives lower collision risk. My results deals with two category of road users bicycling children and motorist. The situation is often very complex, especially in cases with bad interaction. In such cases the level of risk is very high and may remain above the target level of risk according to the theory of risk homeostasis.
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Analysis of Traffic Safety Regarding Public and Individual Transport

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Abstract

Analysis of Traffic Safety regarding Public and Individual Transport

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There are considerable differences between the use of individual and public transport with regard to traffic safety. Until now it has hardly been possible to compare these differences directly with identical relations, due to the fact that especially the routes which are taken by public means of transport mainly consist of several stages for which various means of transport are used (on foot, by bicycle, by car). Therefore, the traffic safety of such a route will be combined with the different traffic safety of each stage, in which different means of transport are used.

A stage model has been developed in one first working step to achieve numerical results allowing to describe all the routes on which at least at one stage a public transport is used. The results are subdivided into individual stages from the complete route.

The application of the stage model and the safety analysis are carried out as follows:
1. From a random sample collection for the whole of the Federal Republic of Germany being an additional collection to KONTIV 82 the route collectives in individual and public transport and their single stages are evaluated in their longitudinal and numerical characteristics respectively. The route collectives differentiate between urban and rural routes and especially commuter traffic.
2. In four sample cases, two urban and two rural regions, questionnaires containing questions about type and length of routes as well as route stages are handed out to passengers of public means of transport. This information is evaluated according to the ones mentioned under point 1.

A safety analysis is carried out from these given sets of data with the help of specific accident rates with regard to the KONTIV data set and with the help of evaluating the accident records including all accidents involving public means of transport in the four above mentioned regions.

The result of this analysis indicates typical accident rates relating to the different kinds of stages used in public means of transport. These rates are confronted with the according accident rates occurred in individual traffic, in which the examination of the single stages does not have the same meaning as it is the case in means of public transport, because in the individual traffic the division of the total route into stages for which different means of transport are used does not appear in the same measure.

The confrontation of the accident rates enables the calculation of the consequences of traffic movements between individual and public traffic taking into consideration the occurrences of accidents, which is shown on the basis of an example.
ANALYSIS OF TRAFFIC SAFETY REGARDING PUBLIC AND INDIVIDUAL TRANSPORT
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University of Kassel

1. INTRODUCTION

It is known that there is a considerable difference of traffic safety between individual and public transport. Until now it was not possible to make a direct comparison with the safety of complete routes, because those routes, especially in which in one stage at least one public transport is used, consist of several part stages with different means of transport (for example on foot, by bicycle, car, bus, train).

A safety comparison of complete routes requires the knowledge of the structure of the routes (the subdivision of the complete routes to single stages), the lengths of the respective stages, the transport production regarding the used means of transport for the single stages as well as their specific accident rates. For the individual traffic more information is known as for of the public transport. Because of this in a first working step initially a route resp. a stage model will be developed, which can describe in detail those routes, in which at least on one stage public transport has been used, according to the used means of transport and the stage length. Moreover, existing research projects will be evaluated as well as additional elevations carried out. An according detailed route model for trips in the individual transport system will not be produced because in this system the subdivision of a whole route into single stages does not point to the same significance.

Due to the evaluation of accident records carried out by chosen transport operators of all accidents involving the participation of public means of transport, in a further working step the typical accident rates of public means of transport are calculated with the help of the route model and indications regarding the transport production, which then are compared in a last working step with the accident rates of individual traffic /1/.

2. ROUTE MODEL OF PUBLIC MEANS OF TRANSPORT

2.1 Data basis

To be able to define more exactly the complete route which is covered by a user of public transport, a route model has been schemed, which illustrates the composition of the whole route resulting from numerous
single stages, especially in those cases in which for the approach to the stop of the public transport system (approach stage) or the leaving stage means of individual transport are used additionally. If the different means of transport which can be used in those stages are taken into account, the results are so called route types, the number growing rapidly according to the increasing number of useable means of transport. The frequency of the single route types as well as their lengths were determined by three different data sets.

The first data set is based on an additional elevation from the random sample elevation in the federal territory of Germany for the purpose of KONTIV 82 /2/, which in 1985 was taken as a subsample elevation from the sample communities of KONTIV 82. Additionally to the written household questionnaire the exact stage structure of all routes was questioned by interviews, to take into consideration also the short stages, which are part of the whole routes. For this additional elevation 911 households from 56 sample communities were drawn and the household members over 10 years of age were questioned. All routes of these household members were evaluated, at least one stage of these routes has been covered with a public transport. The following route collectives were distinguished:

- all routes
- routes in urban areas
- routes in rural areas.

The manner and frequency of the different route types, the length of the approach and leaving stages as well as the length of the public transport stages were evaluated.

This data set, which reflects the behavior of the German federal population, was supplemented through two further data sets, which are derived from two urban traffic zones (the zone of the Mannheimer Verkehrs-AG (MVG), in which 15 tram lines and 18 bus lines transport approx. 920,000 passengers per week, and the zone of the Kasseler Verkehrs-AG (KVG) with 7 tram lines and 16 bus lines and a transport capacity of approx. 750,000 passengers per week) and two zones with a rural structure, namely the Neckar-Odenwald district and the Werra-Meißner district. In the Neckar-Odenwald district were counted 11 bus lines with approx. 99,000 passengers per week, operated by the GBB Rhein-Neckar at the time of the elevation, in the Werra-Meißner district are operated 14 bus lines by the Regionalverkehr Kurhessen (RKH) with approx. 59,000 passengers per week. In all four regions questionnaires were filled out by passengers to obtain the required information.
2.2 Frequency of route types

The evaluation of the frequency of the different route types showed that the majority of routes can only be related to a small number of route types. In table 1, which differs between urban and rural districts, the most frequent route types of the KONTIV-additional elevation are included.

<table>
<thead>
<tr>
<th>frequency in %</th>
<th>route type</th>
</tr>
</thead>
<tbody>
<tr>
<td>all routes</td>
<td>routes in urban area</td>
</tr>
<tr>
<td>53</td>
<td>41</td>
</tr>
<tr>
<td>10</td>
<td>17</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td>5</td>
<td>7</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

( s-rail = suburban railway )

Table 1: Route type frequencies according to KONTIV-additional elevation (only the most represented route types)

Nearly the whole spectrum of all route types can be covered through only 8 route types, whereby for the most part of the route types only one public transport is used. The bus plays clearly the most important role; its significance is distinctly marked more in the rural instead of the urban districts. Route types with two or more changes between different means of transport are not at all among the frequent route types. Furthermore, in none of the frequent route types the car or bicycle is used as an approach or leaving mean of transport.

Table 2 shows the frequencies of the route types, which were evaluated in the four traffic operation areas.

A comparison of the essential results from tables 1 and 2 points out the importance of the bus as the most significant commuter mean of transport in rural areas. In the urban regions on the whole acc. to the KONTIV-data the bus is also the most used transport system followed by the tram, but in the two urban regions the test results proved indeed to invert the two roles.
Table 2: Elevated route type frequencies in urban and rural areas (only the most represented route types)

On the whole these data prove the extraordinary importance of the bus especially in the rural districts. In addition also the railway plays a certain role in the commuter transport. In urban regions – depending on the actually existing transport offer – furthermore the track guided means of transport (tram, underground and suburban railway) prove to be of great importance.

Both tables also show that in the most frequent route types the approach and leaving stages appear to be only on foot.

2.3 Stage length

The analysis of the route types has shown, that the majority of the approach and leaving stages are covered on foot, so these stages are to be described more in detail. In table 3 the stage lengths have been assembled depending on the used mean of transport.

The approach length to the suburban railway and to the rail are clearly above the values for the remaining means of transport. Furthermore, the approach stages to the bus in the rural regions are noticeably over the values for the urban regions. The values of table 3 must be interpreted taking into consideration that in the urban area the railway and in the rural area the tram, underground and suburban railway are only
Table 3: Foot path length (approach stages) depending from the public mean of transport for the various route collectives (indications in km)

<table>
<thead>
<tr>
<th>KONTIV</th>
<th>routes with only 1 bus stage</th>
<th>routes with 2 or more bus stages</th>
<th>tram</th>
<th>underground</th>
<th>sub-urban railway</th>
<th>railway</th>
</tr>
</thead>
<tbody>
<tr>
<td>all ways</td>
<td>0.30</td>
<td>0.23</td>
<td>0.34</td>
<td>0.29</td>
<td>0.75</td>
<td>0.87</td>
</tr>
<tr>
<td>ways in urban area</td>
<td>0.25</td>
<td>0.22</td>
<td>0.35</td>
<td>0.26</td>
<td>0.73</td>
<td>(0.34)</td>
</tr>
<tr>
<td>ways in rural area</td>
<td>0.37</td>
<td>0.25</td>
<td>(0.25)</td>
<td>(0.58)</td>
<td>(0.87)</td>
<td>1.00</td>
</tr>
<tr>
<td>MVG</td>
<td>0.28</td>
<td>0.25</td>
<td>0.28</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>KVG</td>
<td>0.22</td>
<td>0.23</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>GBB</td>
<td>0.35</td>
<td>0.31</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>RKH</td>
<td>0.48</td>
<td>0.42</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

values in brackets: in the random data set only slightly represented

slightly represented in the random data set. A comparison of the approach stage length from the KONTIV-additional elevation for bus stages in urban areas shows quite a good conformity with the according values of the MVG/KVG, just as well as the comparison between the KONTIV values for rural areas and the according values of the GBB, whereas longer approach stages are found in the region of the RKH. The stages to the tram in both urban testing regions are below the KONTIV-average value, due to the relatively dense tram net of these towns.

Remarkable are the differences of the approach stage lengths between the use of only one public mean of transport (bus) and the use of more public means of transport (in this case of more busses). If on the whole route two or more part stages are covered with public means of transport, the passengers do not seem to be prepared to cover the same length for the approach stage as when using only one public mean of transport.

The lengths of the relatively rare approach and leaving stages with bike or car are listed in table 4 but only for the two urban and the two rural testing areas.
Approach resp. leaving means of transport

<table>
<thead>
<tr>
<th></th>
<th>bicycle</th>
<th>car fellow-driver</th>
<th>fellow-passerenger</th>
</tr>
</thead>
<tbody>
<tr>
<td>urban (bus+tram) (MVG+KVG)</td>
<td>1,435</td>
<td>5,136</td>
<td>7,655</td>
</tr>
<tr>
<td></td>
<td>(25)</td>
<td>(29)</td>
<td>(58)</td>
</tr>
<tr>
<td>rural (bus) (GBB+RKH)</td>
<td>1,291</td>
<td>4,239</td>
<td>5,156</td>
</tr>
<tr>
<td></td>
<td>(54)</td>
<td>(20)</td>
<td>(178)</td>
</tr>
</tbody>
</table>

Values in brackets: absolute frequency in the random sample

Table 4: Approach resp. leaving stage length of bicycle and car in the tested regions (indication in km)

The stage lengths for the public transport stages in table 5 show clear differences between the various route collectives.

<table>
<thead>
<tr>
<th></th>
<th>bus</th>
<th>tram</th>
<th>underground</th>
<th>sub-urban railway</th>
<th>railway</th>
</tr>
</thead>
<tbody>
<tr>
<td>all ways</td>
<td>7,30</td>
<td>5,43</td>
<td>4,96</td>
<td>17,98</td>
<td>18,01</td>
</tr>
<tr>
<td></td>
<td>4,27</td>
<td>2,97</td>
<td>7,19</td>
<td>13,23</td>
<td>24,25</td>
</tr>
<tr>
<td>ways in urban area</td>
<td>6,39</td>
<td>4,72</td>
<td>5,10</td>
<td>18,03</td>
<td>(28,01)</td>
</tr>
<tr>
<td></td>
<td>3,15</td>
<td>2,93</td>
<td>7,28</td>
<td>12,93</td>
<td>(21,13)</td>
</tr>
<tr>
<td>ways in rural area</td>
<td>8,21</td>
<td>(2,00)</td>
<td>(3,00)</td>
<td>(17,61)</td>
<td>(16,95)</td>
</tr>
<tr>
<td></td>
<td>6,26</td>
<td>(4,00)</td>
<td>(6,33)</td>
<td>(17,61)</td>
<td>(28,62)</td>
</tr>
<tr>
<td>MVG</td>
<td>4,432</td>
<td>4,870</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>2,946</td>
<td>3,449</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>KVG</td>
<td>5,197</td>
<td>3,910</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>3,820</td>
<td>3,159</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>GBB</td>
<td>9,710</td>
<td>9,680</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>9,710</td>
<td>9,680</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>RKH</td>
<td>14,927</td>
<td>17,912</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1. value: routes with 1 public transport stage
2. value: routes with more public transport stages (... only slightly represented values

The length of a single stage is indicated for the routes with several public transport stages.

Table 5: Public transport stage lengths (indication in km)

The bus stages in rural districts are longer than in urban areas and for the bus as well as for the tram and suburban railway a decrease of the length of the single stages was observed when using several public means of transport in one route.
3. ROUTE MODEL OF INDIVIDUAL MEANS OF TRANSPORT

In the individual traffic no own elevations for the research of route types and stage lengths have been carried out; existing data /3,4/ were used instead. The bicycle traffic was not taken into consideration, not having at the disposal sufficient data information.

For the quite exclusively occurring route type "on foot - stage with individual means of transport - on foot" the average stage lengths were determined as shown in table 6.

<table>
<thead>
<tr>
<th>means of transport</th>
<th>stage length approach + leaving stage</th>
<th>stage with individual transport mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>motor bicycle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>all w. urban</td>
<td>0,132</td>
<td>6,871</td>
</tr>
<tr>
<td>rural</td>
<td>0,080</td>
<td>7,728</td>
</tr>
<tr>
<td>rurals</td>
<td>0,101</td>
<td>7,354</td>
</tr>
<tr>
<td>car (driver)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>all w. urban</td>
<td>0,130</td>
<td>14,125</td>
</tr>
<tr>
<td>rural</td>
<td>0,191</td>
<td>13,215</td>
</tr>
<tr>
<td>rurals</td>
<td>0,102</td>
<td>15,334</td>
</tr>
<tr>
<td>car (fellow urban passenger)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>urban</td>
<td>0,229</td>
<td>18,429</td>
</tr>
<tr>
<td>passenger</td>
<td>0,382</td>
<td>18,328</td>
</tr>
<tr>
<td>rurals</td>
<td>0,120</td>
<td>19,391</td>
</tr>
</tbody>
</table>

Table 6: Stage length in the individual traffic (indication in km)

The length of the stages and the whole route length in the public transport compared with the average route length in the individual traffic proves to be partly much longer. The approach and leaving stages are considerably shorter, they come to little more than one percent of the total route length and therefore, they do not need to be considered in the safety comparison between the individual and public transport.

4. ACCIDENT RATES

4.1 Accident rates regarding the approach and leaving stages in public transport

The accident risk on the approach and leaving stages to and from public transport stops is estimated with the help of specific accident rates for the different means of transport (accident rate = number of injured persons per one million person-km, usually in respect of one year). These specific accident rates for the approach
and leaving stages are shown in table 7. Yet, these accident rates are not subordinated to regional points of view.

<table>
<thead>
<tr>
<th>transport mode during approach and leaving</th>
<th>accident rate (injured p./1 million pkm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>pedestrian</td>
<td>1,4</td>
</tr>
<tr>
<td>bicycle rider</td>
<td>3,8</td>
</tr>
<tr>
<td>car driver</td>
<td>0,4</td>
</tr>
<tr>
<td>fellow passenger</td>
<td>0,6</td>
</tr>
<tr>
<td>motor bicycle rider</td>
<td>12,5</td>
</tr>
</tbody>
</table>

Table 7: Specific accident rates for the approach and leaving stages

4.2 Accident rates for the public transport stages

For the safety analysis regarding the public transport stages the official road traffic accident statistics were used which also include accidents with public means of transport and additionally special accident evaluations in the four testing regions were carried out on the spot.

The accident rates were determined with the help of the official accident statistics for the modes: railway, tram and bus as shown in table 8. In this case it was not possible to include the accident rates of the underground system, because these accidents are not recorded in the official statistics of road traffic accidents. Nevertheless, it must be considered, that also trams have stage sections with detached tracks, and so the accidents occurring on these sections also cannot be taken into consideration when calculating the accident rates.

<table>
<thead>
<tr>
<th>means of transport</th>
<th>accident rate (injured p./1 million pkm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>railway ¹)</td>
<td>0,0185</td>
</tr>
<tr>
<td>tram</td>
<td>0,0969</td>
</tr>
<tr>
<td>bus</td>
<td>0,0539</td>
</tr>
</tbody>
</table>

¹) only Deutsche Bundesbahn (German federal railway)

Table 8: Specific accident rates for the public transport stages

Just as for the accident rates of the approach and leaving stages also these accident rates are not categorized in urban and rural regions. For this reason in all four testing areas (MVG, KVG, GBB, RKH) the internal accident statistics of the transport operators
have been evaluated additionally to be able to obtain specific accident rates, which depend on the categories of urban and rural regions (table 9)\(^*\).

<table>
<thead>
<tr>
<th>means of transport</th>
<th>transport production (mio p-km/ year)</th>
<th>injured persons (injured/year)</th>
<th>accident rate (injured 1 mio p-km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MVG</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tram</td>
<td>165,869</td>
<td>121/21</td>
<td>0,7295/0,1265</td>
</tr>
<tr>
<td>bus</td>
<td>56,279</td>
<td>67/12.5</td>
<td>1,1905/0,2221</td>
</tr>
<tr>
<td>KVG</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>tram</td>
<td>84,200</td>
<td>116.7/20.3</td>
<td>1,3856/0,2415</td>
</tr>
<tr>
<td>bus</td>
<td>53,600</td>
<td>64/12</td>
<td>1,9400/0,2239</td>
</tr>
<tr>
<td>GBB</td>
<td>40,444</td>
<td>0,83</td>
<td>0,0206</td>
</tr>
<tr>
<td>RKH</td>
<td>19,537</td>
<td>0,67</td>
<td>0,0341</td>
</tr>
</tbody>
</table>

\(x_1/ x_2\)

\(x_1 = \) all injured persons registered by the transport operators

\(x_2 = \) only the injured persons registered by the police

(only these values have been used further due to comparative reasons)

Table 9: Traffic data and accident rates of the four testing areas

In fig. 1 the different accident rates from the tables 7 and 8 are put together, in which due to comparative reasons only the registered accidents in the urban testing areas have been included.

It is clearly recognizable that the accident rates in the urban regions are higher than in the rural areas, in which mainly exist the bus and railway systems. On the other hand, in the urban zones the bus is subjected to a higher risk.

The accident rates of the tram show considerable differences between the values on the basis of the official accident statistics and especially the KVG

\(\)* The number of injured persons appearing in the accident statistics of the transport operators is far higher than the ones registered by the police, because accidents occurring without the participation of outsiders (i.e. getting in or out off a tram) normally are not registered by the police.
Fig. 1: Specific accident rates of the public transport modes on the basis of the various accident statistics value. Due to the estimated values (for example the transport production) influencing also the accident rates, at least a part of deviation margin therefore is explainable.

4.3 Accident rates for the whole routes of public transport

From the accident rates of the approach and leaving stages and the values of the public transport stages total accident rates for routes using public means of transport were established, whereby also the frequency of the different stage types was taken into account. This route specific accident rate was calculated as follows:

\[ UR_{wi} = \frac{1}{S_{wi}} \cdot (S_{zi} \cdot UR_{zi} + (\sum_{n=1}^{k} S_{ni} \cdot UR_{ni}) + S_{Ai} \cdot UR_{Ai}) \]  \( (1) \)

with

- \( S_{wi} \) = total length of route from type i
- \( S_{zi} \) = length of the approach stage
- \( UR_{zi} \) = accident rate of the approach route
$S_{ni}$ = length of the public of transport stage n (n = 1, 2, ..., k) of one route

$UR_{ni}$ = accident rate of the public transport stage n with bus, rail, etc.

$S_{Ai}$ = length of the leaving route

$UR_{Ai}$ = accident rate of the leaving route

Evaluating the whole transport production on approach and leaving stages $L_{ZA}$ with equ. (2)

$$L_{ZA} = \sum_{i=1}^{k} \left( H_i \cdot (S_{zi} + S_{Ai}) \right)$$  \hspace{1cm} (2)

with

$H_i$ = frequency of route type i

and evaluating the whole transport production on the public transport stages $L_{OV}$ with equ. (3)

$$L_{OV} = \sum_{i=1}^{k} (H_i \cdot S_{ni})$$  \hspace{1cm} (3)

the average accident rate $UR$ of a region respectively of a specific transport mode can be calculated from equ. (1) to

$$UR = \frac{1}{L_{ZA} + L_{OV}} \sum_{i=1}^{k} \left( H_i \cdot S_{wi} \cdot UR_{wi} \right)$$  \hspace{1cm} (4)

The total accident rates, shown in fig. 2, were put together with equation (4), distinguishing also the routes with and without changes.

An analysis of the values in fig. 2 shows clearly, that the accident risk on the approach and leaving stages is usually higher than on the public transport stages. As the approach and leaving stages according to the analysis of the route length are not neglectfully small, the higher accident rates there have an influence on the accident rate for the total route, i.e. the accident rates of the approach and leaving stages let the total accident rates of those routes in which public means of transport are used to be increased. Furthermore, the analysis showed, that routes with changing the transport mode have lower accident rates than routes without changing, because of the fact that the share of the public transport stage length compared with the share of the approach and leaving stage length in this case is higher than on routes without changes.

On the basis of the KONTIV-data no essential difference is produced between the accident rate in the urban districts and the accident rate in the rural districts, whereas the results from the four testing regions show however a much clearer difference.
Fig. 2: Total accident rates using public transport modes

4.4 Accident rates for the individual traffic

The accident rates for the individual traffic were determined taking into consideration the indications of table 7 and the stage lengths of table 6. Yet, the accident rates for the individual means of transport are not split between urban and rural data collectives. Therefore only total accident rates weighed over the route length, could be calculated for the whole route in the individual traffic, which differ only slightly from the accident rates indicated in table 7, due to the small share of the approach and leaving stages from the total route length. The above mentioned values are shown in fig. 3. Own elevations, as carried out for the public transport, could not yet been envisaged for this research.
5. SAFETY COMPARISON

A comparison of the accident rates in fig. 2 and 3 shows first of all that also taking into account the approach and leaving stages the public transport is safer than the motorized individual traffic. Nevertheless, the safety of total routes, on which at least in one of the stages public means of transport are used, is influenced negatively due to the relatively high risk on the approach and leaving stages. On the whole, nevertheless, the accident risk in the motorized individual traffic for comparable routes is to be found between 2,6 and 6,4 times higher, whereby the lower value stands more for urban regions and the higher value for rural regions.
The following example should point out clearly which consequences the shifting from individual traffic to public transport will have in respect of the traffic safety:

In a town the modal-split will change in a defined relation due to expanding actions in the public transport system from 80 : 20 to 70 : 30 in favour of the public transport. The accident rate \( UR' \) (in respect of the number of injured persons) over all the route types in the public transport should be 0.16 (compare fig. 2), the accident rate \( UR^V \) should be 0.73 (compare fig. 3). The total transport production on this relation should amount to 2 million personkm per year. The shifting from individual traffic to public transport does not lead to any changes on the route lengths and therefore, there are no changes in the transport production. Then the number of injured persons in the previous period has a value of 11.7 in the individual traffic and 0.6 in the public transport. In the later period the number of injured persons changes to 10.2 in the individual traffic and 1.0 in the public transport, i.e. the number of the injured persons on the whole reduces itself by approx. 10%.

In this example the specific accident rates for the single means of transport were not changed at all, inspite of the shifting between individual and public transport. In the range of smaller shifting this is certainly justified. With a greater amount of shifting constant accident rates cannot possibly be taken as a basis. Therefore more research is necessary to find out further exact details.

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Urban Traffic Network Design
A Spatial Approach

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Conventional network design
An urban traffic network can usually be divided into a pedestrian/bicycle network (P/C) and an automobile network. However, a greater part of the P/C network is not separated from the automobile network and is therefore integrated with streets and roads (Figure 1). Within each network and especially for the automobile network the links are classified according to type of area served, speed limits, type of vehicles etc. For the automobile network there are local streets, major streets, freeways etc.

The mixture of traffic generates conflicts between especially the unprotected road users and the automobile traffic. Areas with highly integrated traffic have therefore a large number of serious accidents with pedestrians and cyclists involved. The motor traffic gives also a lot of environmental problems which will be difficult to control.

A Spatial Approach
The highest goal of life must be to free in the widest sense - free from worries, conflicts, disturbances and so on. This means among others that the environment must have such qualities that you can move freely as a pedestrian without any risks, conflicts and disturbances and without any equipment - the highest form of human movement!

At a first approach, we can reach this objective by creating a Free (Foot)Space separated from motor traffic, and by creating
another exclusive space for transport, the (Motor) Transport Space. (Figure 2). The Free Space will include open areas, green belts, parks, playgrounds, sport fields, etc, and foot and cycle paths. The Transport Space will include exclusive links for automobile traffic, freeways, major roads, some feeder streets, etc, but also railway links.

However, it is not realistic to reach a total separation between the Free Space and the Transport Space. At second approach, we therefore have to add an intermediate space, here called the Soft Space between the two other exclusive spaces. (Figure 3). From this figure we can recognise five spaces, namely

1) The Free Space (F)
2) The Transport Space (T)
3) The Soft Space (S), which includes the network for local motor traffic and with special demand on accommodation to the environment and to safety through low volumes, low speed (max 20 mph) and to type of vehicle (no heavy vehicles, no through traffic).
4) The Integrated Free/Soft Space (F/S), between the Free Space and the Soft Space, where pedestrians and cyclists have priority to the street where special physical and legislative arrangements are made in order to identify the area and force the drivers to slow down the speed to a minimum. Also the esthetic qualities are achieved by the street furniture, e.g. pedestrian malls, 'woonerfs'.
5) The Integrated Soft/Transport Space (S/T), between the Soft Space and the Transport Space, where the through-traffic and the local traffic are mixed but have to be accommodated to the conditions of the Soft Space. This means special arrangements by physical and legislative measures for access and speed control (max 30 mph). This Space will only be accepted where the through-traffic cannot be moved outside a residential or a city area.
Urban Traffic Network Design - A Spatial Approach
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Abstract
We need in a city:
1) spaces where you can be together and move freely as a pedestrian/bicyclist
without conflicts and disturbances, the Free Foot Space,
2) spaces where motor transport can be carried out effectively,
the Motor Transport Space.

Through traffic separation we can reach some specific safety and
environmental goals. However, we must accept an integration between the foot
and motor transport functions - under special conditions according to the type
of vehicle, speed etc. Therefore we add an intermediate space, here called
the Traffic Calming Space.

We now can derive five spaces for movement, namely:
1) The Exclusive Foot Space (F), including parks, green belts,
sportgrounds, foot and cycle paths etc,
2) The Integrated Foot Space (F/C)
where the motor traffic is limited to a minimum, and pedestrians and cyclists
have the right of way, e.g. pedestrian malls, woonerfs.
3) The Traffic Calming Space (C),
which include the local traffic network with special demand on accommodation
to the environment and to safety through low volumes, low speed.
4) The Integrated Calming Space (C/T),
where the through-traffic and the local traffic is mixed and has to be
accommodated to the conditions of the Calming Space.
5) The Exclusive Transport Space (T), including local and regional
distributors, freeways, motorways etc, and railways, tramways, busways etc

The strategies for improving the quality of urban life, related to the design of the
urban traffic network, are recommended as:
1. Extend the Foot Space
in order to get more vehicle-free rooms for pedestrians and cyclists so that they
can move, stroll etc without conflicts and disturbances from the motor traffic.
2. Concentrate the Transport Space
to special areas by directing investments to provide links for safe driving and to
protect the environment, even through underground links.
3. Balance the Calming Space
by various means (physical, economic, legislative) to control traffic volume and
speed to acceptable levels according to environmental and safety goals.

The implementation of these strategies - in combination with global measures
to reduce the car dependancy in the society - will favour the safety and
environment not only for all travellers and the city inhabitants in total.
The Conventional Network Design
An urban traffic network is usually divided into a pedestrian/bicycle network and an automobile network. However, a greater part of the pedestrian/bicycle network is not totally separated and therefore pedestrians/bicyclist are mixed the motor traffic on streets and roads (Fig. 1).

The traffic links are classified according to the type of area served, speed limits, type of road users or vehicles, etc. The automobile network is divided into e.g. local streets, major streets, freeways. We have also streets where the automobile is restricted, e.g. on pedestrian malls, woonerfs.

Mixed traffic generates conflicts between the unprotected road users and the automobile traffic and also barriers. It is well known that areas with mixed traffic have a larger accident rate of accidents with pedestrians and cyclists involved than areas with higher degree of separation (OECD 1979). The motor traffic causes also more local environmental problems in areas with imixed traffic than areas with separated traffic. The rate of exhaustes decreases with the distance from the streets and at a distance of 50-100 m, the degree of exhaustes has diminished to a tenth in comparison with the pavement close to the road (Fig. 2).

![Diagram](image)

**Fig. 1.** Conventional classification of an urban network with pedestrian and cycle paths and an automobile network with streets for mixed traffic and some exclusive links for motor traffic.

![Diagram](image)

**Fig. 2.** The degree of exhaustes at various distances. Source: Petersson 1990.
A Spatial Approach
In a city or town you need:
1) spaces where you can be together and move freely as a pedestrian/bicyclist without conflicts and disturbances from surrounding traffic,
2) spaces where motor transport can be carried out effectively.

We can strive to reach this objective by creating:
1) a vehicle-free area for pedestrians where you can walk freely (the highest form of private movement!), the **Free Foot Space (F)**,
2) an exclusive space for motor transport, the **Motor Transport Space (T)**.

The Foot Space includes green belts, parks, squares, playgrounds, sport fields etc and foot paths, may be also cycle paths. The Transport Space will include exclusive links for automobile traffic as freeways, major roads, some feeder streets etc. but also railways, tramways, busways etc and terminals. This exclusive situation is illustrated in Fig. 3. However, it is not realistic to get a total separation between the the Free Space and the Transport Space. We therefore add an intermediate space between these two spaces (Fig. 4), the **Traffic Calming Space(C)**.

We can now derive five spaces (Gunnarsson 1986):
1) **The Exclusive Foot Space (F)** - as mentioned above.

2) **The Integrated Foot Space (F/C)** between the Free Space and the Calming Space, where pedestrians and also cyclists have the right of way in relation to motor vehicles, e.g. pedestrian malls, woonerfs. Special arrangements are made in order to identify the area and slow down the speed to a minimum. Also the aesthetical qualities are achieved by the street furniture. The access to the integrated Foot Space can be limited to only certain hours a day, or to car drivers with a special permission.

3) **The Traffic Calming Space (C)**, which includes the local motor traffic network with special demand on accommodation to the environment and to safety through low volumes, low speed, restricted type of vehicles, no through traffic etc).

4) **The Integrated Calming Space (C/T)**, between the Calming Space and the Transport Space, where the through-traffic and the local traffic are mixed and have to be accommodated to the conditions of the Calming Space. This means special arrangements for speed and access control by physical measures (30 or 50 km/h). This space will be accepted where the through traffic cannot be moved outside a village, a residential or a city area.

5) **The Exclusive Transport Space (T)** as mentioned above.
Fig. 3 The first approach: Separation between Free Foot Space and Motor Transport Space.

Fig. 4. Second approach: Addition of an intermediate space between the Foot Space and the Transport Space.
Establishment of the Free Foot Space

The Free Space can in urban planning be created by:

a) *Land use organisation* so as to provide car free open areas and by localisation of activities so as to avoid conflicts between pedestrian/bicycle traffic and motor traffic (e.g. schools placed so that the pupils do not have to cross streets and roads on their way between the home and the school).

b) *Separation* between pedestrian/bicycle and motor traffic so as to eliminate conflicts at crossings through underpasses or bridges.

c) *Differentiation* of the automobile network in order to direct and concentrate the traffic to the Transport Space and so as to eliminate through traffic, and to accommodate the speed to the road standard and to the environment in general.

The principles, illustrated in Fig. 5, were used in the Swedish SCAF T Guidelines for urban planning with respect to road safety (SCAF T 1968). The first idea of traffic separation in modern age was first applied in 1928 for Radburn, a residential area in New Jersey, U.S.A. (Stein 1957). After the Second War traffic separation and differentiation characterized the construction of new towns in U.K., e.g. Stevenage.

The SCAF T Guidelines were accepted as a planning standard for the design of new residential areas during 1960's and 1970's for Sweden and also other Nordic countries. Follow-up studies have shown that well differentiated and separated areas have a higher degree of safety than areas which are only partly differentiated and separated (SCAF T 1968, OECD 1979). The location of the feeder roads influences the accident rate (SCAF T 1968). If the area is fed from outside, it will have a lower accident rate for esp. children as pedestrians and bicyclists then if it is centrally fed (SCAF T 1968, Bäckström 1982).

In existing areas the Free Space can be extended by:

a) *Relocation of activities* so that more open space is obtained,

b) *Reorganisation of the street network* (grid pattern) in order to provide more pedestrian streets or malls and provide better public transport service, e.g. through traffic cell systems (Fig. 6).

c) *Provision of separated foot and cycles paths*.
Fig. 5. Principles of the SCAFT Guidelines for Urban Planning with Respect to Road Safety. According to: Gunnarsson, Lindström 1970.

1. LOCATING
activities to reduce traffic volumes and conflicts

2. SEPARATING
motor vehicle traffic and pedestrians/cyclists

3. DIFFERENTIATING
the traffic network with respect to function
Fig. 6. Examples of reorganisation of a grid pattern network (a) in order to achieve Free Foot Space and also eliminate through traffic. The traffic cell network, applied in Göteborg is shown in fig. e)
Design of the Integrated Foot Space

The Integrated Foot Space includes pedestrian malls or streets where automobile traffic is reduced to a minimum or only allowed during certain hours for deliveries. The main idea is to keep out through-traffic and restrict the speed with physical obstacles. The Dutch Woonerf model is an example how you can integrate motor traffic under special conditions. This model for integration has been applied in several countries (Fig. 7). However, the implementation cost may be high and still you have conflicts between children and running or parked cars and environmental effects. You can also ban totally private automobiles within the area (with exceptions for delivery cars and the residents). The renewal of the streets will give social and aesthetic positive values for the area which should compensate the reduction in car accessibility.

Fig. 7. Application of the Woonerf model for a residential street.
Design of the Traffic Calming Space

The Traffic Calming Space has to be arranged so that the volume and the speed of the motor traffic is accommodated to the environment. Buchanan (1963) formulated the so called Environmental Capacity as an expression for this approach.

Basically for designing the Traffic Calming Space must be to eliminate the through-traffic. This can be made by offering alternatives routes outside the area in the Transport Space or by arrangements of culs-de-sac. Traffic cell systems can locally improve the environment and the safety (Fig. 6). Other means are tolls, parking fees etc and legislative measures, e.g. special admission only for residents with own parking lots or for disabled persons.

The speed can be reduced, mainly by physical measures or by enforcement. Examples of how physical speed reduction features can be combined are shown in Fig. 8. In a Swedish study, 85% of the drivers lowered the speed to under 30 kmph after humps were installed (Petterson 1979). Installation of humps can reduce the speed up to 20-25 kmph if the distance between the humps is 50-75 metres (Fig. 9).

Figure 8 Examples of speed reduction features. Enlargement of the pedestrian paths in combination with a hump etc in a junction (above) and staggering the alignment in combination with humps on a street section.

Source: Swedish Road Safety Office 1982.
Design of the Integrated Traffic Calming Space (C/T)
A large number of accidents with injuries occurs on major roads. In smaller cities the through-traffic road also serves as a local street and pedestrians and cyclists have to cross the road to reach activities such as schools, shops etc located around the road. Efforts should therefore be made to construct by-passes outside the city. However, the economic possibilities for constructing by-passes are often limited, which means that this type of measure will be delayed or abandoned. A catalogue of ideas for restructuring the roads with narrowings, staggerings, changes in surface, marking, planting etc has been set up by the Danish Road Directorate (1981). The idea is similar to the woonerf model but humps are normally avoided.

An enviromental priority road was introduced through a Danish village in 1985, among others with 'slalom driving' (Fig. 10). Follow-up studies have indicated speed reduction around 10 km per hour at the outer part of the village, 4-5 km per hour in the middle of the village (Danish Road Directorate, 1988).
Fig. 10. Example of Environmental Priority Road. A road through a Danish village before and after reconstruction. Source: Danish Road Directorate 1988.
Analysis of the Motor Traffic Distribution on the Network

Usually we make a quantitative description of the use of the traffic network in terms of vehicles per day or hour, or vehicle kilometres per year or day.

However, we also need a measure which can express the motor traffic loading on the network, here called the Traffic Load Unit.

Traffic Load Unit (TLU) for a road section is proposed to be expressed as:

\[ TLU = (\sum N \times S \times W) \times L \]

where

- \( N \) = Number of vehicles for each type (in ten thousands)
- \( S \) = Speed limit for the street or road section (in km per hour)
- \( W \) = Weight of the vehicles for each type (a private car can be set to a unit \( W=1 \))
- \( L \) = Length of the section (in km)

Example:

A major road, with a length of 10 km and with speed limit of 50 km/h, carries 20,000 vehicles/day. 10% of the cars are trucks (\( W=5 \)).

The Traffic Load Unit per day will be:

\[ TLU = (18 \times 1 + 2 \times 5) \times 50 \times 10 = 14,000 \]

We can determine the loading of motor traffic into the Transport Space and other spaces by calculation of Traffic Load Units. It will be of special interest to study how the traffic distribution can be changed from the loading today all over the streets and roads and to a more directed distribution to a future Motor Transport Space. However, there are no studies available yet on the use of the traffic network, related to the proposed Traffic Load Unit. A calculation of the safety and environmental effects is here also of interest. It is reasonable to assume that 90% or more of the TLU should be directed to the exclusive Transport Space for an efficient use of the traffic network.
RECOMMENDATIONS
The strategies for improving the quality of urban life, related to the design of the urban traffic network, are recommended as:

1. **Extend the Foot Space**
   - in order to get more vehicle-free rooms for pedestrians and cyclists so that they can move, stroll etc without any conflicts and disturbances from the motor traffic. The Foot Space should be connected to city activities and to an attractive public transport system.

2. **Concentrate the Transport Space** to special areas and direct investments to provide links for safe driving and to protect the environment. This can be done by bypasses, outside the cities and larger areas. Even underground links (for transport on road or rail) may be necessary in order to provide conditions for reducing motor traffic on the local level, protect the environment and avoid traffic barriers.

3. **Balance the Calming Space** by various means (physical, economic, legislative) in order to control the traffic volume and the speed to acceptable levels according to environmental and safety goals.

It is my meaning that implementation of these strategies - in combination with global measures to reduce the car dependancy in the society - will favour the safety and environmental factors not only for the pedestrians and cyclists, but also for all travellers and the city inhabitants in total.
References


Comparison of Road Safety in Different Cities

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COMPARISON OF ROAD SAFETY IN DIFFERENT CITIES

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1. Introduction

Securing and increasing the safety in road traffic makes high demands of quality on managing and planning the road safety work on all levels of regional responsibility. For this purpose it is necessary that the level of road safety and its development in the different regions of responsibility (cities and towns, districts, and counties) can be evaluated exactly and comparably and that it can be classified within the total development of the country. Since a comparison of different regions on the basis of absolute facts about the road accident situation doesn't provide a correct picture due to the different prerequisites and data, characteristics were worked out in order to make a comparable evaluation of
- the level of road safety and
- the development of road safety in a region.

These characteristics are
NK - the level coefficient of road safety in a region
    (shortly: level coefficient) and
NI - the level index of road safety in a region
    (shortly: level index)

With the help of these characteristics the level and the development of road safety can be compared on all administrative levels, such as places, towns and cities, districts and counties. On this basis differences of level and development (deficiencies of safety) can be evaluated exactly causing a detailed analysis of reasons within the framework of work on road safety. Applying these characteristics to comparing the road safety among the cities is of special interest.
As about 60 percent of all accidents occur within the municipality decisive measures are necessary for improving the work on road safety particularly on that level. In the following paper basis, initial data, definitions and content of the characteristics will be presented.

2. Basis and initial data

Objective evaluations and comparisons of the road safety in different regions are only possible by means of characteristics representing the relative frequency of accidents.
The relative frequency here is a measure for the hazard that certain accidents will occur with regard to a suitable unit of quantity being the reference unit. Thus characteristics for the comparable evaluation of road safety are composed by the absolute data regarding the accident situation and a reference unit.
To be highly meaningful the initial data regarding the accident situation and the reference unit have to meet, among others, the following demands:
The initial data have to represent the accident situation in the regions exactly and comprehensively; the reference unit must have an important influence on the absolute frequency of the initial data regarding the accident situation.
For this purpose the following initial data regarding the accident situation are included in the characteristics:
- the number of killed people $Z_T$
- the number of injured people $Z_V$
- the number of damaged/destroyed motor vehicles $Z_S$
Via the weights of accident consequence these single data are united into the characteristic of the unit of hazard weight $G$ representing the absolute volume of the accident
situation in a certain region and period of time.

\[ G = Z_T \cdot \varepsilon_T + Z_V \cdot \varepsilon_V + Z_S \cdot \varepsilon_S \]

The weights of accident consequence \((\varepsilon_T, \varepsilon_V, \varepsilon_S)\) indicate the different severity of the accident consequences and are composed by the average costs of the single accident consequences (accident consequence cost) and their relation among each other.

Out of the potential reference units the running performance was chosen. Correlation analyses confirmed that it had the most decisive influence on the absolute frequency of accidents.

The correlation coefficient between the unit of hazard weight \(G\) and the running performance \(F\), calculated according to the following method, is, for instance, \(r = 0.91\) on the level of the GDR counties, thus giving a certainty measure of \(B = 82.8\) percent.

As the running performance can't be evaluated by direct measurements for larger regions and longer periods of time, it will be evaluated as

\[ F = \text{representative total running performance of selected types of motor vehicles in a region (in million vehicle-km) per annum} \]

according to the following initial data:

- number of registered motor vehicles in the region concerned, classified according to the types
- average annual running performance per motor vehicle of the different types.

From the unit of hazard weight \(G\) and the representative total running performance \(F\) the level of hazard \(R\) of the road traffic in a region will be derived by making the quotient.

The level of hazard \(R\) is already a characteristic which indicates the relative volume of the accident situation and which can be used as a comparable measure of road safety.
The aim, however, is to evaluate comparably the road safety exclusively on the basis of regionally specific data. For this purpose all the data which are also efficient for the total GDR have to be eliminated from R. The solution of this problem is to relate the levels of hazard of a region to a unified basic level. As the basic level will be defined the level of hazard of road safety in the country (R_C).

3. Application, definition and content of the characteristics

The level coefficient of road safety (NK in %) for a defined region T will be calculated from the level of hazards R.

\[
NK_T = \frac{R_C}{R_T} \times 100 \text{ in } \%
\]

The level coefficient NK serves for the unified and comparing evaluation of the road safety in the administrative regions. The basis of reference is the average level of hazards of road safety in the country. The level coefficient of road safety NK (in %) indicates how the level of road safety in the region concerned is with regard to the countrywide average level representing 100 percent.

Values of NK:

- **More than** 100 percent mean that the regional level of road safety is above the countrywide average level.
  
  \( N = 100 \) percent means that the regional level of road safety is above that of the country, namely 10 percent.

- **Below** 100 percent means that the regional level of road safety is below the average level of the country.
The level index of road safety (NI in %) indicates the development of the level of road safety in a certain region in a certain year with regard to the average of the past five years.

\[
NI = \frac{NK \cdot 100}{NK_5} \quad \text{in} \% 
\]

The level index NI serves for a unified and comparable evaluation and development of road safety in different regions.

Similarly to the level coefficient NK a basic level will be defined, which the actual values of NK will be related to.

The basic level is the average level of the level coefficient NK of the region concerned in the past five years NK₅. Thus a basic level is available which can be actualized annually and which is in essential free of accidental influences.

The level index of road safety NI (in %) indicates how the level of road safety of the region concerned in the year concerned has developed with regard to the average level of the region in the past five years which represents 100 percent.

Values of NI:
- More than 100 percent mean, that the level of road safety of the region in the year concerned has increased with regard to the average level of the past five years.
- Less than 100 percent mean, that the level of road safety of the region in the year concerned has fallen with regard to the average level of the past five years.

If NI = 93 percent, for example, it can be seen that the level of road safety of the region has fallen with regard to the average level of the past five years, namely by 7 percent.
Divergences from normal value of 100 percent have their reasons essentially in the region concerned. They can be of objective or subjective matter, they can or cannot be influenced. But they always cause the reasons to be found. The values of NK and NI provide a comprehensive evaluation of the level and development of road safety irrespective of:

- the size of the region
- the data, which are also efficient for the total level of the GDR, e.g. the countrywide introduction of safety measures, new legal directions, average volume of traffic and accidents by motorists who are strange in the region etc.

considering

- the decisive characteristics of the accident situation - fatalities, injuries, damaged/destroyed motor vehicles
- the severity of the accident consequences according to unified accident consequence costs per killed and injured people and per damaged/destroyed motor vehicle respectively
- the decisive size / structure of the vehicular stock and of its average annual running performance.

Both characteristics are an initial, decisive step to provide the diagnosis and analysis of differences in the level and development. At the same time they are basic characteristics, on which a whole system of characteristics is based on, which makes possible a very differentiated consideration of the most different factors of influence specific for the region as well as a systematic analysis of the reasons of the accident situation.

Thus the following regionally specific factors of influence on the level of safety can already be shown, considered and eliminated respectively:

- the structure of the road network
- traffic from outside the region which is above or below the average
- the density of the road network above or below the average
- the running performances of different types of traffic means in the vehicular stock
- particular situations in road traffic with extraordinary accident consequences.

Table 1 shows a real example of application for the level coefficient NK, table 2 for the level index NI, taken from the statistic annual report of the traffic police in the county of Dresden.
Table 1

Level coefficient of road safety for the regions of the county Dresden
(in %, related to the GDR)

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<td>105.14</td>
<td>118.68</td>
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<td>95.48</td>
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<td>123.21</td>
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<td>105.81</td>
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<tr>
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<td>93.14</td>
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<td>116.44</td>
<td>93.49</td>
<td>114.59</td>
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<td>89.21</td>
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<td>88.74</td>
<td>90.00</td>
<td>90.02</td>
<td>94.12</td>
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</table>
Level index of road safety for the regions of the county Dresden (in %, related to the GDR)

<table>
<thead>
<tr>
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<td>95.65</td>
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<td>Meißen</td>
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<td>104.19</td>
<td>106.67</td>
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<tr>
<td>Riesa</td>
<td>110.36</td>
<td>109.00</td>
<td>107.14</td>
<td>92.96</td>
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<td>109.77</td>
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<td>county</td>
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<td>102.98</td>
<td>101.14</td>
<td>105.29</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Effects of Speed Reducing Measures in Danish Residential Areas

Ulla Engel
Senior Research Scientist
Danish Council of Road Safety Research
Denmark
SAFETY EFFECTS OF SPEED REDUCING MEASURES IN DANISH RESIDENTIAL AREAS.

To types of streets were monitored, play-areas and quiet-streets. In play-areas the speed of the cars are restricted to 15 km/h and in quiet-streets to 30 km/h.

Within this part of the project 44 experimental streets, i.e. 5 play-areas and 39 quiet streets, and 52 control streets have been observed. The experimental group contains 30 km of streets and the control group contains 35 km of streets.

The research design is a before—after study with a control group. The before and after periods were three years each, individually chosen for each experimental street.

The main effect was a significant change in the number of casualties in the experimental streets of -16 with confidence limits ranging from +6 and -38 casualties. This effect is due to the experiment: The change of the status of the streets from ordinary streets (general speed limit of 50 km/h) to play-areas and quiet-streets.

Due to a reduction in traffic which may possibly be caused by the change in street status, there was furthermore a significant change in the number of casualties of -7 with confidence limits ranging from -5 to -10 casualties.

There has also been a significant change in the rate of casualties (= number of casualties per road user km) of -72% with confidence limits ranging from -4% to -92%, due to the change in street status.

Also speed measurements were conducted in all experiment streets and selected control streets.

The speed of about 22,000 cars were measured and the results will be compared to the accident analysis.
EFFECTS OF SPEED REDUCING MEASURES IN DANISH RESIDENTIAL AREAS

Ulla Engel
Senior research scientist
Danish Council of Road Safety Research
Denmark

1. BACKGROUND

The Danish Road Traffic Act was revised in 1976. In that connection a new code 40 was applied:

"Local reorganisation of traffic. On the recommendation of the local authorities the Minister for Justice may, in concert with the Minister for the Environment, deviate from the rules laid down in part 2, 3 and 4 of this Act, to the extent required in order to promote an expedient traffic reorganisation in local areas".

The safety consequences of this new code have been studied since 1982.

According to this code, which came into force on 1st November 1978, it was made legal under certain circumstances to change the status of streets from "traffic streets", i.e. streets with priority for vehicles, to "living areas" with priority for pedestrians.

The implementation of "living areas" demanded the implementation of different physical measures, among others a speed limit of 15 km/h, cfr. figure 1.

![Figure 1.](image)

However, in order to meet demands from the public as well as the local authorities, a second type of "living areas" was created. Here the demands were a speed limit of 30 km/h, cfr. figure 1, and some, but fewer restrictions concerning physical measures. The costs of the implementations of this street-type were much lower compared to the costs of the implementations of 15 km/h streets.

It were the 30 km/h-streets, which became popular and most frequent especially in existing living areas. As a
paradox it has to be mentioned that the 30 km/h-streets involved no change in the traditional priority for vehicles. Hence the evaluation is mainly based on a street-type which was only an indirect outcome of code 40 in the Road Traffic Act.

2. MAIN PARAMETERS

The evaluation of effects is based on two main parameters: Accident frequency and motor vehicle speed. The research design in both studies is a before-and-after study with control group, and the before and after periods were three years each in the accident study.

2.1 Accident frequency

Two different studies concern the accident frequency. A total of 729 experimental streets has been investigated "before and after" the changes in status (i.e. local reorganisation) were implemented. From this study we learned about the currency of the local reorganisation of streets in Denmark in the period 1.11.1978 - 31.7.1983, and about the changes in the number of traffic accidents per road km. (Engel & Thomsen, 1989a).

Simultaneously a second study was performed. A total of 44 experimental streets and 52 control streets were investigated more intense. The group of experimental streets consisted of the "local reorganisation activity" in Denmark in the period of one year 1.8.1982 - 31.7.1983.

The control streets have been selected one by one in order to match each of the experimental streets. From this study we learned about the changes in risk (i.e. number of traffic accidents per road-user km). (Engel & Thomsen, 1989b).

2.2 Motor vehicle speed

On 41 of 44 experimental streets and 13 of 52 control streets the speed of motor vehicles was recorded.

No significant reduction in the mean speed on the control streets was found from the before to the after period. Hence further studies were only based on data from the experimental streets. The speed of all together 8,504 motor vehicles is included in this part of the study.

The speed measurements were collected continuously over a distance of up to 200 m. Hence it was possible to observe changes in "speed profiles" before and after the implementation of physical "speed reducing" measures. (Engel & Thomsen, 1990).
3. RESULTS

3.1 Accidents per road km

This part of the project concerns 10 km of 15 km/h - streets and 223 km of 30 km/h - streets. The control group was all urban streets in Denmark belonging to the local government authorities, all together 18,935 km of streets.

The accidents and casualties are related to the length of each street. No traffic figures were available for this retrospective study.

The main effect was a significant change in accidents in 30 km/h - streets of -24% (i.e. -77 accidents in three years), cfr. table 1. In the same street type there was a change in casualties of -45% (i.e. -88 casualties in three years), cfr. table 2.

Table 1. Changes in accidents per road km on 50 15 km/h-streets and 679 30 km/h-streets distributed on "inner areas"(1) and "outer areas"(2).

| change/ | change            | change            |
| street type | without control group | including control group |
| inner area(1) | 182,92 | 186,49 |
| outer area(2) | 34,21 | 35,93 |
| inner area(1) | -65,19 | -64,76 |
| outer area(2) | -26,44 | -25,54 |

(1) "inner area", i.e. the part of the street limited by the speed-limit sign.
(2) "outer areas", i.e. parts of the street located just outside the "inner area".
Indices yields upper and lower limits of the corresponding 95 per cent confidence intervals.

VTI RAPPORT 363A
Tabel 2. Changes in casualties per road km on 50 15 km/h-streets and 679 30 km/h-streets distributed on "inner areas"(1) and "outer areas"(2).

<table>
<thead>
<tr>
<th>change/ street type</th>
<th>change without control group</th>
<th>change including control group</th>
</tr>
</thead>
<tbody>
<tr>
<td>inner area(1)</td>
<td>-100%</td>
<td>-100%</td>
</tr>
<tr>
<td>outer areas(2)</td>
<td>-2,23</td>
<td>21,44</td>
</tr>
<tr>
<td>15 km/h streets</td>
<td>-39,92%</td>
<td>-25,40%</td>
</tr>
<tr>
<td>30 km/h streets</td>
<td>-43,27</td>
<td>-29,52</td>
</tr>
<tr>
<td>outer area(1)</td>
<td>-56,04%</td>
<td>-45,42%</td>
</tr>
<tr>
<td>areas(2)</td>
<td>-63,08</td>
<td>-54,17</td>
</tr>
<tr>
<td>30 km/h streets</td>
<td>-65,94</td>
<td>-57,73</td>
</tr>
</tbody>
</table>

(1) "inner area", i.e. the part of the street limited by the speed-limit sign.
(2) "outer areas", i.e. parts of the street located just outside the "inner area".

Indices yields upper and lower limits of the corresponding 95 per cent confidence intervals.

In areas close connected to the 30 km/h streets there were a change in accidents of -18% (i.e. -150 accidents in three years) and in casualties of -21% (i.e. -106 casualties in three years).

These are the results after the trends in the control group have been taken into consideration.

No significant changes were found in the 15 km/h-streets.

3.2 Accidents per road user km

This part of the project contains 5 15 km/h streets and 34 30 km/h streets, all together 30 km of experimental streets. The control group contains 52 streets, about 35 km all together.

The accidents and casualties are related to the number of road user km travelled in each street, one day in the pe-
periods 6 a.m. - 10 a.m. and 12 a.m. - 4 p.m.

The main effect is a significant change in the number of casualties per road user km of -72% with confidence limits ranging from -4% to -92%, due to the change in street status, cfr. table 3.

Table 3. Change in number of casualties per road user km in "inner areas" and "outer areas" of 44 experiment streets.

<table>
<thead>
<tr>
<th>type of area</th>
<th>inner areas</th>
<th>outer areas</th>
</tr>
</thead>
<tbody>
<tr>
<td>casualties per road user km</td>
<td>-4%</td>
<td>+583%</td>
</tr>
<tr>
<td></td>
<td>-72%</td>
<td>+96%</td>
</tr>
<tr>
<td></td>
<td>-92%</td>
<td>-44%</td>
</tr>
</tbody>
</table>

Super- and subindeks are upper and lower limits in a 95 per cent confidence interval.

Furthermore there has been a significant change in the number of seriously injured of -78% with confidence limits ranging from -26% to -93%, cfr. table 4 and 5.

Tabel 4. Casualties on 44 experimental streets (inner areas) distributed on degree of severity and time period.

<table>
<thead>
<tr>
<th>degree of severity/time period</th>
<th>experimental streets inner areas</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>killed</td>
</tr>
<tr>
<td>before</td>
<td>0</td>
</tr>
<tr>
<td>after</td>
<td>1</td>
</tr>
<tr>
<td>total</td>
<td>1</td>
</tr>
</tbody>
</table>

All figures mentioned above are results after the trends in the control group have been taken into consideration.
Table 5. Casualties on 52 control streets (inner areas) distributed on degree of severity and time period.

<table>
<thead>
<tr>
<th>degree of severity/time period</th>
<th>control streets</th>
<th>inner areas</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>killed</td>
<td>serious injuries</td>
</tr>
<tr>
<td>before</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>after</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>total</td>
<td>1</td>
<td>18</td>
</tr>
</tbody>
</table>

3.3 Motor vehicle speed

30 km/h streets must be "equiped" in a way which discourage motor vehicle drivers from driving of speeds exceeding 30 km/h. To obtain such behaviour speed reducing measures have to be implemented with a mutual distance of a maximum of 100 metres.

Table 6 exposes the preliminary results of speed measurements taken at the center of the countermeasures as well as at distances of 50 metres away from the center. Results are shown for each of eight types of countermeasures.

From the table it can be seen, that the greatest change in speed is caused by humps even in a distance of 50 metres. The lateral dislocation and the narrowing of the carriageway produces more ambiguous results.

This analyses is performed on the basis of all together 36 theoretical combinations of the three types of countermeasures even though several of these combinations do not exist.

Finally a model has been constructed. Speed data is taken from the study based on the 36 theoretical combinations of the three types of countermeasures. Moreover road geometry have been added.
Table 6. Changes in speed in three sections (-50 metres, 0, +50 metres) for humps, lateral dislocations and narrowing of the carriageway) with 95% confidence limits.

<table>
<thead>
<tr>
<th>countermeasure</th>
<th>change in speed</th>
<th>95% confidence limits</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>km/h</td>
<td></td>
</tr>
<tr>
<td>- 50 metres</td>
<td></td>
<td></td>
</tr>
<tr>
<td>hump, circelsegment</td>
<td>-13.7</td>
<td>-17.6 -9.8</td>
</tr>
<tr>
<td>hump, elevated junction</td>
<td>-13.7</td>
<td>-16.5 -10.9</td>
</tr>
<tr>
<td>hump, plateau and</td>
<td>-6.5</td>
<td>-12.7 -0.3</td>
</tr>
<tr>
<td>circlesegment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>hump, plateau</td>
<td>-26.8</td>
<td>-32.1 -21.5</td>
</tr>
<tr>
<td>lateral dislocation, single</td>
<td>-12.1</td>
<td>-15.5 -8.7</td>
</tr>
<tr>
<td>lateral dislocation, double</td>
<td>-3.7</td>
<td>-7.0 -0.4</td>
</tr>
<tr>
<td>narrowing of carriageway</td>
<td>-1.7</td>
<td>-8.7 5.3</td>
</tr>
<tr>
<td>0 metre, center of countermeasure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>hump, circelsegment</td>
<td>-7.9</td>
<td>-10.5 -5.3</td>
</tr>
<tr>
<td>hump, elevated junction</td>
<td>-14.7</td>
<td>-17.4 -12.0</td>
</tr>
<tr>
<td>hump, elevated area before junction</td>
<td>3.2</td>
<td>-10.5 16.9</td>
</tr>
<tr>
<td>hump, plateau and</td>
<td></td>
<td></td>
</tr>
<tr>
<td>circlesegment</td>
<td>-21.2</td>
<td>-26.1 -16.3</td>
</tr>
<tr>
<td>hump, plateau</td>
<td>-8.1</td>
<td>-10.3 -5.9</td>
</tr>
<tr>
<td>lateral dislocation, single</td>
<td>4.1</td>
<td>1.8 6.4</td>
</tr>
<tr>
<td>lateral dislocation, double</td>
<td>0.8</td>
<td>-1.4 3.0</td>
</tr>
<tr>
<td>narrowing of carriageway</td>
<td>-3.4</td>
<td>-8.1 1.3</td>
</tr>
<tr>
<td>50 metres</td>
<td></td>
<td></td>
</tr>
<tr>
<td>hump, circelsegment</td>
<td>-3.6</td>
<td>-22.2 -15.0</td>
</tr>
<tr>
<td>hump, elevated junction</td>
<td>-8.3</td>
<td>-24.2 7.6</td>
</tr>
<tr>
<td>hump, plateau and</td>
<td></td>
<td></td>
</tr>
<tr>
<td>circlesegment</td>
<td>-23.6</td>
<td>-47.3 0.1</td>
</tr>
<tr>
<td>hump, plateau</td>
<td>-16.8</td>
<td>-27.0 -6.6</td>
</tr>
<tr>
<td>lateral dislocation, single</td>
<td>3.2</td>
<td>-14.2 20.6</td>
</tr>
<tr>
<td>lateral dislocation, double</td>
<td>-9.5</td>
<td>-28.2 9.2</td>
</tr>
<tr>
<td>narrowing of carriageway</td>
<td>-14.1</td>
<td>-23.2 -5.0</td>
</tr>
</tbody>
</table>

With a chosen level of significance of 1 per cent 8 variables are important for the reduction of speed, cfr. table 7.
Table 7. Significance probability for explanatory variables in the final model for speed change from "before" to "after".

<table>
<thead>
<tr>
<th>significance probability</th>
<th>short description of variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0001</td>
<td>$\bar{V}_f$ mean speed, before</td>
</tr>
<tr>
<td>0.0001</td>
<td>$h_1$ height of hump</td>
</tr>
<tr>
<td>0.0001</td>
<td>$x^2$ distance to primary countermeasure, squared</td>
</tr>
<tr>
<td>0.0001</td>
<td>$a_1$ distance to &quot;last countermeasure&quot;</td>
</tr>
<tr>
<td>0.0001</td>
<td>F type of lateral dislocation</td>
</tr>
<tr>
<td></td>
<td>(single, double)</td>
</tr>
<tr>
<td>0.0001</td>
<td>I street narrowing</td>
</tr>
<tr>
<td>0.0007</td>
<td>$a_2$ distance to &quot;next countermeasure&quot;</td>
</tr>
<tr>
<td>0.3630</td>
<td>x distance to primary countermeasure</td>
</tr>
</tbody>
</table>

In table 8 estimates of speed reduction for each of the eight variables in the model are given, and below an example of calculation of speed reduction on the basis of the estimates is exposed.

Figure 2 shows the speed profiles recorded in the before-period and in the after-period for the street Kongens Enge in the municipality of Greve. From the before- to the after-period the street was change into a 30 km/h street.

First we will look at the recorded speed change at section -10 which is located about 41.5 metres before the center of the countermeasure, a hump combined with narrowing of the carriage-way, measured in the direction of driving (from left to right).

Recorded mean speed "before" 57.57 km/h
Recorded mean speed "after" 25.48 km/h

Recorded change -32.09 km/h

Secondly we will calculate the speed reduction at the same section (-10) by the mean of our model.

The parameters in the model is already given in table 8. We just have to add the following information:
Mean speed before (at section -10) 57.57 km/h
Distance from -10 to the center of the hump 41.50 metres
Distance from the center of the hump to the "last countermeasure" 61.50 metres
Distance from the center of the hump to the "next countermeasure" 36.50 metres
Height of hump 0.12 metres

Speed change calculated with the model:

\[
\begin{align*}
- 0.6451 \times 57.57 \\
+ 0.003760 \times 41.5 \\
+ 0.0005352 \times 41.5^2 \\
- \ln(1 + 1/61.5) \times 148.3189 \\
- \ln(1 + 1/36.5) \times 81.5011 \\
- 1.001 \times 12 \\
- 4.680 \\
+ 29.058 \\
\end{align*}
\]

= - 28.29 km/h

This gives us the following result:

<table>
<thead>
<tr>
<th>Recorded mean speed &quot;before&quot;</th>
<th>57.57 km/h</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calculated mean speed &quot;after&quot;</td>
<td>28.29 km/h</td>
</tr>
</tbody>
</table>

Calculated change -29.28 km/h

In this example the results from the recorded change in speed and the calculated change differ with 0.21 km/h. In general disparities of 5-10 km/h are not seldom which is illustrated by the models degree of explanation of 61 per cent.
Table 8. Parameter-estimates for the final model.

<table>
<thead>
<tr>
<th>short description of variables</th>
<th>estimate</th>
<th>95 per cent confidence limits</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \bar{V}_f ) mean speed, before</td>
<td>-0.6451</td>
<td>-0.6859 -0.6043</td>
</tr>
<tr>
<td>( x ) distance to the primary countermeasure</td>
<td>0.003760</td>
<td>-0.004337 0.01186</td>
</tr>
<tr>
<td>( x^2 ) distance to the primary countermeasure, squared</td>
<td>0.0005352</td>
<td>0.0004658 0.0006046</td>
</tr>
<tr>
<td>( a_1 ) distance from the primary to the &quot;last&quot; countermeasure</td>
<td>-148.32</td>
<td>-195.59 -101.05</td>
</tr>
<tr>
<td>( a_2 ) distance from the primary to the &quot;next&quot; countermeasure</td>
<td>-81.50</td>
<td>-128.50 -34.50</td>
</tr>
<tr>
<td>( h_1 ) height up to hump</td>
<td>-1.001</td>
<td>-1.0984 -0.9036</td>
</tr>
<tr>
<td>( F ) type of lateral dislocation: single</td>
<td>-2.017</td>
<td>-3.131 -0.9025</td>
</tr>
<tr>
<td>double</td>
<td>-4.724</td>
<td>-5.871 -3.577</td>
</tr>
<tr>
<td>none</td>
<td>-0.0</td>
<td>km/h</td>
</tr>
<tr>
<td>( I ) street narrowing: exists</td>
<td>-4.680</td>
<td>-6.322 -3.039</td>
</tr>
<tr>
<td>none</td>
<td>0.0</td>
<td>km/h</td>
</tr>
<tr>
<td>constant</td>
<td>29.058</td>
<td>26.38 31.74</td>
</tr>
</tbody>
</table>

* In this expression for "distance", the distance is measured in metres.
Figure 2
3.4 Accidents per road user km in relation to motor vehicle speed

For all together 40 experimental streets data concerning accidents, traffic flow and motor vehicle speed are available for a before-period as well as an after-period.

So 80 sets of observations have constituted the basis for the creation of models showing connections between on one side the number of accidents and the number of casualties and on the other side explanatory variables in this case mean speeds, traffic flow (road user km) and the number of intersection per road km (intersection-density).

The results are exposed in three groups equivalent to number of casualties (c), number of accidents with personal injuries (p) and number of accidents with damage only (d). The mean speed is designated with (s), traffic flow with (t) and intersection-density with (i).

\[ c = 0.5249 \times 10^{-6} \times s^{1.627} \times t^{1.083} \]
\[ p = 1.5770 \times 10^{-6} \times s^{1.362} \times t^{0.939} \]
\[ d = 0.4220 \times 10^{-6} \times s \times t^{1.635} \]

In all three cases both traffic flow and intersection-density show a positive influence on the number of accidents and the number of casualties respectively. The mean speed, however, seems to have no influence on the damage only accidents.

In all three cases the value of the exponent to the traffic flow is one; hence it is possible to devide with the traffic on both sides of the sign of equation. The three formulae now contain information on risk i.e. accidents/casualties per road user km.

The parameter-estimates for the three models are given in table 9 together with the upper and lower limits of the corresponding 95 per cent confidence interval.

The influence of the different variables on the number of accidents and casualties respectively is shown in figures 3-10, where figures no 3-5 correspond to the model for casualties, no 6-8 correspond to the model for personal injury accidents and no 9-10 correspond to the model for damage only accidents.

Figure no 3 exposes the estimated influence of speed on the number of casualties in the 40 experiment streets. Figure no 4 exposes the estimated influence of traffic flow (road user km) on the number of casualties and figure no 5 exposes the estimated influence of intersection-density. The curvature represents only observed values.
Tabel 9. Parameter- and interval-estimates of the models describing casualties, personal injury accidents, and damage only accidents.

<table>
<thead>
<tr>
<th>model and variables</th>
<th>parameter- and interval-estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td>number of casualties (c)</td>
<td></td>
</tr>
<tr>
<td>speed (s)</td>
<td>2.583</td>
</tr>
<tr>
<td></td>
<td>1.627</td>
</tr>
<tr>
<td></td>
<td>0.6705</td>
</tr>
<tr>
<td>intersection-density (i)</td>
<td>1.450</td>
</tr>
<tr>
<td></td>
<td>1.083</td>
</tr>
<tr>
<td></td>
<td>0.7165</td>
</tr>
<tr>
<td>number of personal injury accidents (p)</td>
<td></td>
</tr>
<tr>
<td>speed (s)</td>
<td>2.377</td>
</tr>
<tr>
<td></td>
<td>1.362</td>
</tr>
<tr>
<td></td>
<td>0.3467</td>
</tr>
<tr>
<td>intersection-density (i)</td>
<td>1.370</td>
</tr>
<tr>
<td></td>
<td>0.939</td>
</tr>
<tr>
<td></td>
<td>0.5078</td>
</tr>
<tr>
<td>number of damage only accidents (d)</td>
<td></td>
</tr>
<tr>
<td>intersection-density (i)</td>
<td>1.968</td>
</tr>
<tr>
<td></td>
<td>1.635</td>
</tr>
<tr>
<td></td>
<td>1.302</td>
</tr>
</tbody>
</table>

Indices yield upper and lower limits of the corresponding 95 per cent confidence limit.

Figure no 3 for example, shows the influence of mean speed in the interval from 12 km/h to 59 km/h and the factor with influence on the number of casualties is moving within the interval of 56-752 casualties.

The same figure exposes the effects of a change in the mean speed e.g. from 50 km/h to 30 km/h. The corresponding factors for casualties is 581 and 253 respectively, and the estimated change in the number of casualties per road user km is -56 per cent.

The other figures can be "read" in a similar way.
Figure 3  Mean Speed versus Casualty Factor

Figure 4  Traffic Flow versus Casualty Factor
Figure 5 Junction Density versus Casualty Factor

Figure 6 Mean Speed versus Accident Factor (Personal Injury Accidents)
**Figure 7** Traffic Flow versus Accident Factor (Personal Injury Accidents)

**Figure 8** Junction Density versus Accident Factor (Personal Injury Accidents)
Figure 9  Traffic Flow versus Accident Factor
(Damage Only Accidents)

Figure 10  Junction Density versus Accident Factor
(Damage Only Accidents)
References

Engel, U. & L.K. Thomsen
§40 gaders sikkerhed
En analyse af politirapporterede ulykker på danske §40 gader baseret på ulykkesfrequenser før hhv. efter ændring af gadestatus.
(The safety of code 40 streets. An analyses of police reported accidents on Danish code 40 streets based on accidents per road km before and after the change in street status)
Danish Council of Road Safety Research
Documentation Report 1/1989(a) Copenhagen

Engel, U. & L.K. Thomsen
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En analyse af politirapporterede trafikulykker på §40 gader og 52 kontrolgader baseret på ulykkesfrequenser før hhv. efter ændring af gadestatus.
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(The safety of code 40 streets. An analyses of the speed of cars on 41 code 40 streets and 13 control streets before and after the change in street status)
Danish Council of Road Safety Research
Documentation Report 2/1990 Copenhagen
A Case Study Evaluating Traffic Warning Devices with Respect to Operating Speeds and Accident Rates

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A CASE STUDY EVALUATING TRAFFIC WARNING DEVICES
WITH RESPECT TO OPERATING SPEEDS AND ACCIDENT RATES

by

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Abstract

During the period from 1940 to 1970, the only direct safety criterion available to highway engineers in the geometric design guidelines of most Western European countries and the U.S. was (still is) directed toward evaluating driving dynamic aspects, such as calculating for a given design speed minimum radii of curves, superelevation rates, necessary stopping sight distances, minimum radii of crest vertical curves, etc.

Since the 1970s, two other indirect design criteria, with respect to traffic safety, were additionally provided in the geometric design guidelines of some European countries. For instance, German, Swedish and Swiss designers were partially provided with design criteria to insure design consistency between design elements, and to harmonize design speed and operating speed.

These three criteria should always constitute the basis for achieving good design levels, and for detecting poor design levels. Between the two, good and poor design levels, lies fair design. Criteria for defining these three design levels have been provided by the authors in several research reports and publications, based on reliable boundaries for changes in degrees of curve, curvature change rate (Kurvigkeit), and operating speeds between successive design elements, and boundaries for differences between operating speed and design speed at a certain curved section. While new designs should always correspond to good design levels, at least those roadway sections which correspond to poor design levels -- where a safety problem was documented -- have normally been upgraded to good design levels. But, this was not possible to conduct for a large part of the existing low-volume, two-lane road network, which corresponds to fair design levels, because of a lack of financial resources. This part of the road network, which was built between 20 and 35 years ago, was based on a design speed concept which allowed to build in inconsistencies in horizontal alignment, such as between the flatter and sharper portions of the highway when the controlling horizontal curves correspond sometimes to an improperly selected design speed. In these cases, transition sections which would require from the driver unexpected speed changes, which may in turn lead to hazardous driving maneuvers, may exist. These road sections exhibit at least minor inconsistencies in geometric design. Normally, correcting the existing alignment is not necessary since low cost projects like traffic control devices may, to a certain extent, be successful in correcting these defects.

Traffic control devices are classified into three functional groups: Warning devices, Regulatory devices, and Guiding devices.
This study was undertaken to determine how TRAFFIC WARNING DEVICES (for example: posted recommended speed plates) do relate to operating speeds and accident rates on curves/curved sections of two-lane rural highways. A total of 322 road sections were considered for the study. For each of the curved sections, the following design parameters were collected: posted recommended speeds, degree of curve, curvature change rate (Kurvigkeit), length of curve, superelevation rate, gradient, sight distance, lane width, shoulder width, and average annual daily traffic.

It should be noted that degree of curve is an important U.S. design parameter. It is similar to the German design parameter, curvature change rate. In the case of a single design element (tangent or curve), degree of curve and curvature change rate are comparable.

Regression analyses indicated that posted recommended speed, degree of curve, or curvature change rate were able to explain most of the variation in operating speeds and accident rates on two-lane rural highways. The other parameters, with the exception of superelevation rate, contributed a little to the strength of the relationships.

In addition, the study revealed that with increasing degree of curve, or curvature change rate (1) the gap between operating speed, posted recommended speed, and design speed becomes wider and wider, and (2) accident rates show an increase too despite the stringent recommended speeds, often combined with arrow designations and chevrons, installed at curved sites.

The potential usefulness of the relationships set forth in the study is that they provide the design engineer with regression models for (1) predicting operating speeds and accident rates on curved sections of two-lane rural highways, based on actual posted recommended speed plates, and (2) determining, from an actual knowledge of degree of curve or curvature change rate of a certain roadway section, what recommended speed plates, if any, should be posted. It provides the engineer with an idea on where to draw the line between good, fair and poor designs, based on changes in operating speeds and accident rates. It should be noted that studies have shown that the accident rate in the case of fair design is always double that for good design, despite the posted traffic warning devices, but still a lot better than that for poor design.
A CASE STUDY EVALUATING TRAFFIC WARNING DEVICES WITH RESPECT TO OPERATING SPEEDS AND ACCIDENT RATES

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1. INTRODUCTION

During the period from 1940 to 1970, the only direct safety criterion available to highway engineers in the geometric design guidelines of most Western European countries and the U.S. was (still is) directed toward evaluating driving dynamic aspects, such as calculating for a given design speed minimum radii of curves, superelevation rates, necessary stopping sight distances, minimum radii of crest vertical curves, etc., see for example [1, 2]. Proper application of these guidelines in the design of horizontal and vertical alignments, and cross section would have made the use of traffic control devices, at least on low and intermediate traffic volume two-lane rural highways, partially unnecessary.

However, recent knowledge about driving behavior, as well as research experience about unfavorable design, as related to accident spots and dangerous road sections, require more and more a better understanding of the design speed concept on which the guidelines are based. For instance, studies have shown that the design speed concept allows to build in critical inconsistencies in horizontal alignment, such as between the flatter and sharper portions of the highway when the controlling horizontal curves correspond sometimes to an improperly selected design speed. In these cases, transition sections may exist which would require from the driver unexpected critical speed changes that may in turn lead to hazardous driving maneuvers [3-7]. Furthermore, studies have shown that the driving behavior on an observed road section, for example a curved section, often exceeds by substantial amounts the design speed on which the original design of the road section was based, especially at lower design speed levels [4,5,7-13].

Since the 1970s, two other indirect design criteria, with respect to traffic safety, were additionally provided in the geometric design guidelines of some European countries. For instance, German, Swedish and Swiss designers were partially provided with design criteria to insure design consistency between design elements, and to harmonize design speed and operating speed [8,10,11].

These criteria should always constitute the basis for achieving good design levels, and for detecting poor design levels. Between the two, good and poor design levels, lies fair design. Criteria for defining these three design levels have been provided by the authors in several research reports and publications, based on reliable boundaries for changes in degree of curve [14], curvature change rate (Kurvigkeit) [8], and operating speeds between successive design elements, as well as boundaries for differences between operating speed and design speed at a certain curved section [15-22]. While new designs should always correspond to good design levels, at least those roadway sections which correspond to poor design levels - where a safety problem was documented - have normally been upgraded to good design levels. But, this is not possible to conduct for a large part of the existing low-volume, two-lane road network in many countries, which corresponds to fair design levels, because of a lack of financial resources. Those road sections exhibit at least minor inconsistencies in geometric design. Normally, correcting the existing alignment is not necessary since low cost projects like traffic control devices may, to a certain extent, be successful in correcting these defects.

The main functions of traffic control devices are similar in most countries. Based on the U.S. regulations, they are discussed in the following, for example [23,24]:

Traffic control devices provide the road user with information regarding performance requirements along his route of travel. Traffic control devices may either supplement or modify the basic rules of the road, and this information must be transmitted by appropriate devices at
the specific time or location. Devices are classified into three functional groups as follows:
- **Regulatory devices** give the road user notice of traffic laws or regulations that apply at a
given place or on a given roadway. Disregard of such devices is punishable as an in-
fraction, violation, or misdemeanor.
- **Warning devices** call attention of the road user to conditions, on or adjacent of the road-
way, that are potentially hazardous to traffic operations.
- **Guiding devices** provide directions and information to the road user regarding route
designations, destinations, distances, roadway delineation, points of interest, and
other geographical or cultural information.

In this research investigation, only the impact of warning devices, more precisely posted
recommended speeds (for example, advisory speed plates combined or not combined with
arrow designations and/or chevrons) at curved sites will be examined to determine their effect
on operating speeds and accident rates on two-lane rural highways.

2. DATA COLLECTION AND REDUCTION

From spring 1984 to spring 1986, State routes throughout northern New York State were
investigated, and a sufficient number of road sections, normally consisting of sequences of tan-
gent-to-curve (or curved section) -to-tangent, were located on rural two-lane highways. The se-
lection process resulted in 261 road sections of varying degrees of curve (database I). The
sections selected were distributed by lane width as follows: 12-ft lanes (3.6 m, 84 sections), 11-ft
lanes (3.3 m, 92 sections), and 10-ft lanes (3.0 m, 85 sections). The average section length was
about 1 mile (1.6 km); thus nearly 300 miles (500 km) of State route sections were investigated
and approximately 30.000 miles (50.000 km) were driven during the selection phase. The road
sections selected provided the widest range of changes in horizontal alignment that could be
found by observation or by actual information obtained from the regional offices of the New
York State Department of Transportation (NYSDOT). The selection process attempted to
maintain a regional distribution, and at the same time retain the longest roadway segments
[17,18].

For each of the selected sections, the following data were collected: degree of curve (degrees
per 100 ft), curvature change rate (degrees per half-mile), length of curve (ft), superelevation
rate (percent), gradient (less than or equal to 5 percent), sight distance (miles), lane width (ft),
shoulder width (ft), average annual daily traffic (between 400 and 5000 vehicles per day), traffic
warning devices (arrow signs, recommended speed plates, and chevrons), a 3-year accident data
and operating speed data were used in this study. Normally, the speeds of 60 to 80 passenger
cars [17,18,24,25] under free flow conditions were measured in each direction of traffic for
determining the operating speed (expressed by the 85th-percentile speeds); for example in a
curved - or straight road section.

Because the accident sample (a total of 815 accidents) is not large enough to allow
disaggregation into several categories, only the total number of accidents was analyzed in the
study. To assess the quality of the road, the accident rate that is defined by the number of acci-
dents per 1 million vehicle-miles was chosen to be used. The accident rate for each of the 261
investigated road sections was calculated as follows:

\[
\text{ACCR} = \frac{(\text{No. Acc.}) \cdot (10^6)}{(365) \cdot (\text{No. Years}) \cdot (\text{LC}) \cdot (\text{AADT})}
\]

Where:
- **ACCR** = Number of accidents per 1 million vehicle-miles,
- **No. Acc.** = Number of accidents in the curved section from January 1982 to December 1984
  (3 years), related to all vehicle types,
- **No. Years.** = Number of years investigated (i.e., 3 years),
- **LC** = Length of curve or curved section (miles),
- **AADT** = Average Annual Daily Traffic (vehicles per day, both directions).

The reader who is interested in a detailed discussion of the data collection and reduction
process should consult [17, 18, 20-22, 28, 30].
3. OUTCOME OF THE DATA ANALYSES

In the following the main objective of discussion will be to investigate the effect of traffic warning devices and additional design- and traffic volume parameters on operating speeds and accident rates of two-lane rural highways. But first of all the impact of the German design parameter "Curvature Change Rate (CCR)" [8, 26] and the American design parameter "Degree of Curve (DC)" [2, 14] on operating speeds will be analysed, compared and evaluated, as basis for the following investigations.

The results of the analysis process will be discussed in the order outlined below:
1. One-to-one relationship between curvature change rate and operating speed.
2. One-to-one relationship between degree of curve and operating speed.
3. Example applications for both methods (curvature change rate and degree of curve).
4. Effect of traffic warning devices and additional parameters on operating speeds and accident rates.
5. Effect of degree of curve and additional parameters on operating speeds and accident rates.

3.1 One-to-one relationship between curvature change rate and operating speed

Extensive investigations in Germany and in addition by the authors in the U.S.A [5, 7-9, 15, 16, 26, 27] have shown that speed characteristics on horizontal alignments can be adequately described in terms of the curvature change rate design parameter CCR, defined as the absolute sum of the angular changes per section length of roadway with similar road characteristics. A section is any length of horizontal alignment that exhibits similarity in road characteristics, such as similar radii of curves and similar cross sections.

Mathematically, curvature change rate can be expressed corresponding to the German Guidelines for the Design of Roads [5, 7-9, 26] as:

\[
CCR = \frac{\sum |\omega_i|}{L}
\]  

\[ L = \text{total length (ft) of the road section, that exhibits similarity in road characteristics,} \]
\[ \omega_i = \alpha_i + \tau_j \text{(degrees),} \]
\[ \alpha_i = L_i/R_i \times (57.3) \text{ (degrees, circular curve),} \]
\[ \tau_j = L_j/2R_i \times (57.3) \text{ (degrees, transition curve),} \]
\[ L_i = \text{Length of circular curve i (ft),} \]
\[ L_j = \text{Length of transition curve j (ft),} \]
\[ R_i = \text{Radius of curve i (ft).} \]

It then follows that

\[
CCR = \frac{\sum |\frac{L_i}{R_i}| + \sum |\frac{L_j}{2R_i}| - 1 \times (57.3) \times 2640}{L} \]

for sections with circular and transition curves (where, 2640 is the conversion factor to convert to units of degrees per half-mile).

To examine the relationship between the German design parameter curvature change rate and operating speed, regression analysis was used. In conducting regression analyses, the following steps were carried out:
1. Determine the regression coefficients through the use of least squares procedures;
2. Test the statistical significance of each regression coefficient to see if it should remain in the model; and
3. Determine the overall predictive accuracy of the final model to determine how well it predicts the dependent variable of interest; in our case operating speed.
Based on the above stipulations, the resulting regression equations between curvature change rate (CCR) and 85th-percentile speed for passenger cars, by lane widths, are as follows [15, 18]:

- **Lane width 10 ft:** \( V_{85} = 55.132 - 0.042 \text{ CCR} \); \( R^2 = 0.846 \); SEE = 2.753 mph (4)
- **Lane width 11 ft:** \( V_{85} = 57.602 - 0.040 \text{ CCR} \); \( R^2 = 0.731 \); SEE = 2.721 mph (5)
- **Lane width 12 ft:** \( V_{85} = 59.515 - 0.038 \text{ CCR} \); \( R^2 = 0.836 \); SEE = 2.261 mph (6)

where:
- \( V_{85} \) = estimate of 85th-percentile speed (mph),
- \( \text{CCR} \) = curvature change rate (degrees per half-mile),
- \( R^2 \) = coefficient of determination, and
- SEE = standard error of estimate.

As can be seen, there is a strong correlation between 85th-percentile speed and CCR for all lane widths as implied by the large coefficient of determination \( R^2 \). From these equations, contrary to the curvilinear relationships found in Germany [8, 26], the driving behavior in New York State related to CCR is described by linear relationships. For the second-order term of a curvilinear regression equation, the T-test yielded nonsignificant results at the 95 percent level of confidence for all lane widths.

The linear regression equations are plotted in Figure 1, which shows the relationship between operating speed and CCR for different lane widths. The potential usefulness of the graph is obvious. By knowing the lane width and difference in CCR between two road sections with similarities in horizontal alignment the expected change in operating speed between the two sections can be predicted. For example, if a roadway with lanes 10 ft wide has a tangent section with CCR = 0, followed by a curved section with CCR = 400 degrees per half-mile, the expected change in operating speed is approximately 55 - 39 = 16 mph. Furthermore, for an observed road section, differences between the selected design speed and the expected operating speed can be predicted in the early design or redesign stages.

In order to use the curvature change rate procedure, it must be clear as to what is meant by homogeneous sections, or similarity in road characteristics. While such a section can be defined as having similar design elements for the horizontal alignment, it still can be difficult to comprehend this concept. Graphical techniques were developed to identify subsections with similarities in horizontal alignment in the German Geometric Design Standards [8] to alleviate this problem.

However, for a large part of the existing low-volume, two-lane rural network in many countries it is usually difficult to find road sections of appreciable lengths with similarities in horizontal alignment than for newer more curvilinear ones. In these cases, dividing the road into uniform subsections results mainly in analyzing each curve individually. For single curves, the curvature change rate formula reduces to just the degree of curve formula, as it was derived by the authors in a recent article about "New Ideas for the Design of Two-Lane Rural Roads", published in the International Technical Journal: Straßen- und Tiefbau, FRG, Vol 5 and 6, 1989 [22].

### 3.2 One-to-one relationship between degree of curve and operating speed

Degree of curve of a given circular curve is the angle (or number of degrees) subtended at the center by an arc of 100 ft. length [14]. It is defined as degrees per 100 ft. as shown in the illustration opposite. Contrary to many countries, which consider radius of curve, or curvature change rate as important design parameters, highway geometric design in the U.S. is mainly related to the design parameter "degree of curve" [14]. The relationship between degree of curve and radius of curve is given by the following equation:

\[
DC = \frac{5729.6}{R_i} \quad \text{(degrees per 100 ft.)}
\]

where \( DC \) = degree of curve, and \( R_i \) = radius of curve i (ft). It follows:

\[
R_i = \frac{5729.6}{DC} \quad \text{(ft.)} \quad \text{or} \quad R_i = \frac{1746.3}{DC} \quad \text{(m)}
\]

Conversion factor: 1 ft. = 0.3048 m.
Based on the stipulations previously mentioned, the resulting regression equations between degree of curve (DC) and 85th-percentile speed for passenger cars, by lane widths, are as follows:

- **Lane width 10 ft:** \( V_{85} = 55.646 - 1.019 \text{ DC} \); \( R^2 = 0.753; \text{SEE} = 3.485 \text{ mph} \) (8)
- **Lane width 11 ft:** \( V_{85} = 58.310 - 1.052 \text{ DC} \); \( R^2 = 0.746; \text{SEE} = 2.646 \text{ mph} \) (9)
- **Lane width 12 ft:** \( V_{85} = 59.746 - 0.998 \text{ DC} \); \( R^2 = 0.824; \text{SEE} = 2.344 \text{ mph} \) (10)

where DC is measured in degrees per 100 ft.

A plot relating 85th-percentile speed to DC is shown in Figure 2. By knowing the change in DC between two successive curves or between a curve and a tangent, the expected changes in operating speed can be predicted in the same way as with CCR. Again, the T-test showed nonsignificant results at the 95 percent level of confidence for all lane widths for the second-order term of a curvilinear regression equation. For comparison reasons radii of curve in the dimensions "feet" and "meter" are given corresponding to Equations 7c and 7d in Figure 2, also.

Comparison of design parameters CCR and DC related to operating speeds shows that similarly valid results can be achieved by using DC. For the present database, Figures 1 and 2 show that estimates of the 85th-percentile speed can be achieved by applying either of these design parameters without committing significant error when related to single curves or curved sections.

Because the design, in many countries, especially for two-lane rural roads, mainly consists of sequences of tangents and curves (or curved sections), the parameter DC will be used for future recommendations to detect critical operating speed changes in horizontal alignment. In the case of more curvilinear alignments, like in Germany, the CCR method can be used, also.

### 3.3 Example applications for both methods

To determine the change in operating speed by using the previously discussed methods, Figure 3 shows a typical road section consisting of a curve and two long tangents located on State Route 19, in open country in New York State. The pavement width is 22 ft, lane width 11 ft, with additional shoulders of 3 ft on both sides. Pavement conditions were good, whereas shoulder conditions were fair. On the approaches to the curve from both sides, advisory speed plates of 25 mph (40 km/h) combined with arrow sign designations were installed. In addition, the curved section was equipped with chevrons.

By applying Figure 1 for CCR, one obtains the following expected operating speed for an 11-ft lane:
- **Tangent:** CCR = 0; \( V_{85} = 58 \text{ mph} (93 \text{ km/h}) \),
- **Curve:** CCR = 401 degrees per half-mile; \( V_{85} = 42 \text{ mph} (67 \text{ km/h}) \),
- **Speed Change:** \( \Delta V_{85} = 16 \text{ mph} (26 \text{ km/h}) \).

By applying Figure 2 for DC, one obtains the following expected operating speeds for an 11-ft lane:
- **Tangent:** DC \( = 0^\circ \); \( V_{85} = 58.5 \text{ mph} (94 \text{ km/h}) \),
- **Curve:** DC \( = 15.2^\circ \); \( V_{85} = 42.5 \text{ mph} (68 \text{ km/h}) \),
- **Speed change:** \( \Delta V_{85} = 16 \text{ mph} (26 \text{ km/h}) \).

The observed 85th-percentile speed for both directions on the curve was \( V_{85} = 40 \text{ mph} (64 \text{ km/h}) \); the observed 85th-percentile speed for both directions on the tangent was \( V_{85} = 57.5 \text{ mph} (92 \text{ km/h}) \); and the resulting observed speed change was \( \Delta V_{85} = 17.5 \text{ mph} (28 \text{ km/h}) \).

Thus, for the example application, several conclusions could be drawn:
- Actually measured 85th-percentile speeds agree well with the expected operating speeds of Figures 1 and 2 for the tangents and curve.
- The DC method leads to the same results as the CCR method.
- The expected and the actually measured operating speed exceed the posted recommended speed at the curved site, decisively.

Thus, it is interesting to note that in spite of the applied stringent traffic warning devices, the expected change in the speed profile is still 16 mph (26 km/h) based on Figures 1 and 2, which agrees well with the actually measured change of 17.5 mph (28 km/h). These speed changes are far larger than the maximum allowable speed changes for good and fair design, suggested by the authors in Table 1, on the basis of T-test results of accident rates for changes in degree of curve classes between successive design elements [18, 20, 28].
The observed changes in degree of curve $\Delta DC = 15.2^0$ and changes in operating speed $\Delta V85 = 16$ mph (26 km/h) imply that the horizontal alignment of Figure 3 corresponds to "Poor Design Practices" - "Case 3 of Table 1" - and a high cost project, for example a redesign, should be recommended.

In this connection, one should note that according to Figure 3 even very stringent traffic warning devices (for example, posted recommended speeds of $25$ mph = 40 km/h combined with arrow signs and chevrons) did not prove to be effective in correcting strong dissimilarities in horizontal alignment, for example by smoothing the speed profile in such a way that less

---

**Figure 1:** Relationship between 85th-Percentile Speed and Curvature Change Rate for Passenger Cars for Individual Lane Widths

<table>
<thead>
<tr>
<th>Curve Change Rate Rate (degrees per half-mile)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1: $V85 = 55.132 - 0.042 CCR, R^2 = 0.846$</td>
</tr>
<tr>
<td>2: $V85 = 57.602 - 0.040 CCR, R^2 = 0.731$</td>
</tr>
<tr>
<td>3: $V85 = 59.515 - 0.038 CCR, R^2 = 0.836$</td>
</tr>
</tbody>
</table>

---

**Figure 2:** Relationship between 85th-Percentile Speed and Degree of Curve (Radius of Curve) for Passenger Cars for Individual Lane Widths

<table>
<thead>
<tr>
<th>Radius of Curve (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2000</td>
</tr>
<tr>
<td>6000</td>
</tr>
</tbody>
</table>

Legend: $1$ mile $= 1.609$ km, $1$ ft $= 0.3048$ m, $1$ mph $= 1.609$ km/h
Date: Oct. 9, 1985
Time: 4:30 p.m.
Weather Condition: dry

Pavilion

State: New York
Route Number: 19
Section Length: 1.1 mile
Lane Width: 11 ft (~3.30 m)
Degree of Curve: 15.2°
Radius of Curve: 375 ft (~115 m)
Curvature Change Rate: 401 degrees per half-mile
Grade: 0 %
Shoulder Width: 3 ft (~1.00 m)
Superelevation Rate: 7 %
Recommended Speed: 25 mph, (40 km/h)
with arrow signs and chevrons
AADT: 1700 vehicles per day
Measured 85th-percentile Speed: 40 mph (64 km/h)
(Average for both directions)

Figure 3: Case Study of STATE ROUTE 19

CASE 1 (GOOD DESIGN):
Range of change in degree of curve: \( \Delta DC \leq 5^\circ \).
Range of change in operating speed: \( \Delta V_{85} \leq 6 \text{ mph (10 km/h)} \).
For these road sections, consistency in horizontal alignment exists between successive design elements, and the horizontal alignment does not create inconsistencies in vehicle operating speeds.

CASE 2 (FAIR DESIGN):
Range of change in degree of curve: \( 5^\circ < \Delta DC \leq 10^\circ \).
Range of change in operating speed: \( 6 \text{ mph} < \Delta V_{85} \leq 12 \text{ mph (20 km/h)} \).
These road sections may represent at least minor inconsistencies in geometric design between successive design elements. Normally, they would warrant traffic warning devices, but no redesigns.

CASE 3 (POOR DESIGN):
Range of change in degree of curve: \( \Delta DC > 10^\circ \).
Range of change in operating speed: \( \Delta V_{85} > 12 \text{ mph (20 km/h)} \).
These road sections have strong inconsistencies in horizontal geometric design between successive design elements combined with those breaks in the speed profile that may lead to critical driving maneuvers. Normally redesigns are recommended.

Table 1: Recommended Ranges for Good, Fair and Poor Design Practices between Successive Design Elements [18, 20, 28].
extensive breaks in the operating speeds between successive design elements do exist. In those cases (Case 3 of Table 1: $\Delta V_{85} > 12 \text{ mph} = 20 \text{ km/h}$) increased critical driving maneuvers have to be expected and accident black spots may originate.

The same results can be found on the basis of the German Guidelines for the Design of Roads (RAS-L-1), 1984 [8], which only allow a maximum speed change of $\Delta V_{85} \leq 10 \text{ km/h} (6 \text{ mph})$ in the operating speeds between successive design elements, as well as related to the Swiss Standard [11], where the maximum speed change of $\Delta V_{85} \leq 20 \text{ km/h} (12 \text{ mph})$ should not be exceeded.

But as already previously mentioned, it is not possible to upgrade the large part of the insufficiently designed, low-volume, two-lane rural network in many countries to good design levels, because of a lack of financial resources. Therefore, the authors introduced corresponding to Table 1 "The fair design level", where to a certain extent minor inconsistencies in geometric design can be alleviated by the proper use of traffic warning devices, for example for restoration, rehabilitation or resurfacing strategies.

In order to achieve this object the impact of traffic warning devices on operating speeds and accident rates will be analysed.

### 3.4 Effect of traffic warning devices and additional parameters on operating speeds and accident rates

As noted in Refs. 17, 18, 22, 28 and 30, there are many factors which may have a possible effect on operating speeds and accident rates. These include, but are not limited to, important site characteristics like degree of curve (DC), traffic warning devices (for example, advisory speed plates or recommended speeds: RS, etc.), length of curve (LC), superelevation rate (SE), gradient (G), sight distance (SD), lane width (LW), shoulder width (SW), and average annual daily traffic (AADT).

For the evaluation of the quantitative effects of these independent variables, the multiple linear stepwise regression technique (MAX $R^2$ improvement technique) was used [18, 29, 30]. The stepwise technique consists of adding one independent variable to the regression equation in each step. Thus, the stepwise process produces a series of multiple regression equations, in which each equation has one independent variable more than its predecessor in the series of regression equations.

The following stipulations were used to terminate the stepwise process and determine the final multiple regression equation: (1). The selected equation has to have a multiple regression coefficient $R^2$, which is significant at the 0.05 level; (2) Each of the independent variables included in the multiple regression equation has to have a regression coefficient which is significantly different from zero at the 0.05 level; and (3) None of the independent variables included in the multiple regression equation are highly correlated with each other.

The reader who is interested in a further discussion of the multiple linear regression technique should consult [17, 18, 30]. These studies have shown that the most successful parameters in explaining much of the variability in operating speeds and accident rates on two-lane rural highways were degree of curve and posted recommended speed. The impact of degree of curve on operating speeds and/or accident rates has already been reported in several publications [20-22, 28].

Therefore, the impact of traffic warning devices (expressed by recommended speed limits) on operating speeds and accident rates shall be discussed in the following.

#### 3.4.1 Operating speed vs. recommended speed, for passenger cars

Based on the stipulations, previously mentioned, the following regression models between operating speed, recommended speed and various design and traffic volume parameters were obtained for individual lane widths:

**Lane width 10 ft: Overall Equation**

\[
\begin{align*}
V_{85} &= 27.965 + 0.414 (RS) + 0.001 (LC) ; R^2 = 0.588; \text{SEE} = 4.516 \text{ mph} \\
\end{align*}
\]

(11)

As can be seen from Equation 11, the design parameters shoulder width (SW), sight distance (SD), gradient (G), and average annual daily traffic (AADT) were not included in the model because the regression coefficients associated with these parameters were not significantly
different from zero at the 95 percent confidence level. Superelevation rate SE was not included in the predictive regression equation because it was highly correlated with recommended speed (RS).

However, comparing Equation 11 with the following equation, which includes only the design parameter RS, shows that the effect of length of curve (LC) amounts to only 3.2 percent of the variation in the estimated operating speeds.

Reduced Equation:
\[ V_{85} = 27.173 + 0.459 \ (RS) ; R^2 = 0.556; \text{SEE} = 4.675 \text{ mph} \]  
This small standard error (4.675) and moderate \( R^2 \) value (0.556) suggest that the relationship represented by Equation 12 is a strong competitor to that represented by Equation 11.

Similarly, the resulting overall and reduced regression equations for passenger cars, for 11 ft and 12 ft lanes, are as follows:

**Lane width 11 ft: Overall Equation**
\[ V_{85} = 29.264 + 0.428 \ (RS) + 0.313 \ (SW) + 0.0002 \ (SD) + 0.0005 \ (AADT) \]
\[ R^2 = 0.785; \text{SEE} = 2.455 \text{ mph} \]  
**Reduced Equation**
\[ V_{85} = 29.190 + 0.479 \ (RS); R^2 = 0.744; \text{SEE} = 2.654 \text{ mph} \]

**Lane width 12 ft: Overall Equation**
\[ V_{85} = 26.544 + 0.562 \ (RS); R^2 = 0.835; \text{SEE} = 2.268 \text{ mph} \]  
**Reduced Equation**
\[ V_{85} = 26.544 + 0.562 \ (RS); R^2 = 0.835; \text{SEE} = 2.268 \text{ mph} \]

Equations 12, 14 and 16 are depicted in Figure 4a. The figure indicates that with increasing recommended speed the operating speed is increasing, too.

### 3.4.2 Accident rate vs. recommended speed, for all vehicle types

Between accident rate, recommended speed and various design and traffic volume parameters the following regression models were obtained for individual lane widths:

**Lane with 10 ft: Overall Equation**
\[ A_{CCR} = 38.102-0.625 \ (RS) - 0.009 \ (SD) - 0.003 \ (AADT) \]
\[ R^2 = 0.288; \text{SEE} = 13.029 \text{ acc/mvm} \]  
As can be seen from Equation 17, the design parameters shoulder width (SW), length of curve (LC), and gradient (G) were not included in the model because the regression coefficients associated with these parameters were not significantly different from zero at the 95 percent confidence level. However, comparing Equation 17 with the following equation, which includes only the design parameter RS, shows that the effect of sight distance (SD) and AADT amounts to only 3.9 percent of the variation in the estimated accident rates.

**Reduced Equation**
\[ A_{CCR} = 41.922-0.689 \ (RS);R^2 = 0.249;\text{SEE} = 13.299 \text{ acc/mvm} \]  

The small coefficient of determination (0.249) and the relatively large standard error (13.299 acc/mvm) in Equation 18 are not at all surprising because accident research relationships are not simple and direct ones, but are often complex, and changes in frequency of accidents are often the result of many factors in addition to the design parameters and traffic volume data. Similarly, the resulting regression equations for all vehicle types, for 11 ft and 12 ft lanes, are as follows:

**Lane with 11 ft: Overall Equation**
\[ A_{CCR} = 31.461 - 0.365 \ (RS) - 0.003 \ (LC) - 0.002 \ (AADT) \]
\[ R^2 = 0.308; \text{SEE} = 6.722 \text{ acc/mvm} \]  

VTI RAPPORT 363A
Reduced Equation
ACCR = 29.024 - 0.425 (RS); $R^2 = 0.252$; SEE = 6.945 acc/mvm

Lane with 12 ft: Overall Equation
ACCR = 35.773 - 0.539 (RS) - 0.0009 (LC) - 0.0009 (AADT)
$R^2 = 0.731$; SEE = 3.373 acc/mvm

Reduced Equation
ACCR = 33.145 - 0.556 (RS); $R^2 = 0.693$; SEE = 3.578 acc/mvm

Figure 4: Nomogramm for Evaluating Operating Speeds and Accident Rates as Related to Recommended Speed for Individual Lane Widths

LEGEND: 1 mile = 1.609 km, 1 ft = 0.3048 m, 1 mph = 1.609 km/h
Equations 18, 20 and 22 are depicted in Figure 4b. The figure indicates that with increasing recommended speed the accident rate is decreasing. This is not at all surprising, since less stringent traffic warning devices (that means higher recommend speeds) will be posted at lower degree of curve classes, where the accident situation can be evaluated as less serious, see Table 2 and Figure 5b.

3.5 Effect of degree of curve and additional parameters on operating speeds and accident rates

Research which evaluated the impact of design parameters (degree of curve, length of curve, superelevation rate, lane width, shoulder width, sight distance, gradient) and traffic volume on 261 curved sections of two-lane rural highways in the State of New York, has demonstrated that the most successful parameters in explaining the variability in operating speeds and accident rates was "degree of curve (DC)",[17, 18, 20, 22, 28, 30] and "posted recommended speed (RS)" as discussed in chapters 3.41 and 3.42. All the other design and traffic volume parameters, helped the regression models, but contributed little to the strength of the relationships. The corresponding equations and graphical illustrations, as related to degree of curve, are shown in Table 2 and Figure 5. It is interesting to note that the reduced regression Equations 23, 25 and 27 for the relationship between operating speed and degree of curve correspond to the one-to-one relationship of chapter 3.2, compare Figure 2 and Equations 8, 9 and 10, also.

As can be expected, Table 2 and Figure 5 reveal, with increasing degree of curve (decreasing radius of curve), that operating speed (V85) is decreasing while the accident rate is increasing.

<table>
<thead>
<tr>
<th>1: 10-ft. lanes (about 3 m):</th>
<th>2: 11-ft. lanes (about 3.3 m):</th>
<th>3: 12-ft. lanes (about 3.6 m):</th>
</tr>
</thead>
<tbody>
<tr>
<td>V85 = 55.646 - 1.019DC; R² = 0.753</td>
<td>V85 = 58.310 - 1.052DC; R² = 0.746</td>
<td>V85 = 59.746 - 0.998DC; R² = 0.824</td>
</tr>
<tr>
<td>ACCR = -1.023 + 1.513DC; R² = 0.300</td>
<td>ACCR = -0.257 + 1.375DC; R² = 0.462</td>
<td>ACCR = -0.546 + 1.075DC; R² = 0.726</td>
</tr>
</tbody>
</table>

V85 = Estimate of the operating speed, expressed by the 85th-percentile speed for passenger cars (mph), Conversion Factor: mph = 1.609 km/h.

DC = Degree of curve (degrees / 100 ft.), range: 0° to 27°

Conversion Factors: ft = 0.3048 m,
R = 5729.6/DC (ft) or R = 1746.3/DC (m),
where R = radius of curve,

ACCR = Estimate of accident rate for all vehicle types (acc./106vehicle-miles), range: 1° to 27°
Conversion Factor: vehicle-mile = 1.609 vehicle-kilometer,
R² = Coefficient of determination.

Table 2: Reduced Regression Equations for Evaluating Operating Speeds and Accident Rates as Related to Degree of Curve for Individual Lane Widths [21].

A cross-validation of the models containing degree of curve or posted recommended speed on an additional sample of 61 rural, curved roadway sections, (database II) of varying degrees of curve, revealed that the regression models could be used with a moderate degree of confidence for prediction purposes [30].
4. DISCUSSION OF THE RESULTS

With respect to the established relationships, the following comprehensive statements can be made:

(1) With decreasing posted recommended speeds (RS) operating speeds (V85) are decreasing according to Figure 4a.

Normally, operating speeds are higher than recommended speeds. The speed differences between the two become wider and wider as recommended speeds decrease, see the following listing (lane width 11 ft):

<table>
<thead>
<tr>
<th>RS mph</th>
<th>50</th>
<th>45</th>
<th>40</th>
<th>35</th>
<th>30</th>
<th>25</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>V85 mph</td>
<td>53</td>
<td>51</td>
<td>48</td>
<td>46</td>
<td>44</td>
<td>41</td>
<td>39</td>
</tr>
<tr>
<td>RS-V85 mph</td>
<td>3</td>
<td>6</td>
<td>8</td>
<td>11</td>
<td>14</td>
<td>16</td>
<td>19</td>
</tr>
</tbody>
</table>

Figure 5: Nomogramm for Evaluating Operating Speeds and Accident Rates as Related to Degree of Curve (Radius of Curve) for Individual Lane Widths [21]
(2) With decreasing posted recommended speeds accident rates increase according to Figure 4b. This may be due to the fact that drivers do not take into consideration seriously that normally low recommended speeds are posted at those critical curved sites, where high degrees of curve do exist, see issue (3) and Figure 6. It may also be largely due to a lack of driving experience or unsafe vehicle, compare the following listing (lane width 11 ft):

<table>
<thead>
<tr>
<th>RS</th>
<th>mph</th>
<th>50</th>
<th>45</th>
<th>40</th>
<th>35</th>
<th>30</th>
<th>25</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACCR</td>
<td>acc./10^6 veh.-miles</td>
<td>7.8</td>
<td>9.9</td>
<td>12.0</td>
<td>14.1</td>
<td>16.3</td>
<td>18.4</td>
<td>20.5</td>
</tr>
</tbody>
</table>

Issues (1) and (2) reveal that the wider the gap becomes between operating speeds and recommended speed limits, the higher are the expected accident rates.

(3) With increasing degree of curve operating speeds decrease according to Figure 5a, but not in such a way that they coincide with the posted recommended speeds, as issue (1) reveals, see the following listing (lane width 11 ft):

<table>
<thead>
<tr>
<th>DC deg./100ft.</th>
<th>0</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>V85 mph</td>
<td>58</td>
<td>53</td>
<td>47</td>
<td>43</td>
<td>37</td>
<td>32</td>
</tr>
<tr>
<td>RS* mph</td>
<td>55**</td>
<td>50</td>
<td>40, 35</td>
<td>30, 25, 20</td>
<td>&lt; 20</td>
<td></td>
</tr>
</tbody>
</table>

* Recommend speed, estimated by comparing the 85th-percentile speeds of listing (1) and (3).
** 55 mph, overall speed limit in the U.S.A.

The above listing reveals that up to a degree of curve of 5°, which represents normally good design practices (compare Table 1) the difference between operating speed and posted recommended speed is, more or less, negligible. However this difference reaches up to 12 mph (20 km/h) for 10° of curve and exceeds normally this margin for degrees of curve DC > 10°. Thus, the findings of this research suggest that for high degrees of curve the actual operating speeds (V85) are definitely higher than the posted recommended speed limits.

(4) Therefore, it is not surprising that according to Figure 5b with increasing degree of curve accident rates increase also, as they do with decreasing posted recommended speeds, compare issue (2) and the following listing (lane width 11 ft):

<table>
<thead>
<tr>
<th>DC deg./100ft.</th>
<th>1*</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACCR acc./10^6 veh.-miles</td>
<td>1.1</td>
<td>6.6</td>
<td>13.5</td>
<td>20.4</td>
<td>27.2</td>
<td>34.1</td>
</tr>
</tbody>
</table>

* range: 1° to 27°

For instance, a comparison between degrees of curve 5° (R = 350m) and 10° (R = 175m), reveals already an accident rate on the latter twice as high as on the former. For 15° (R = 115m) this comparative ratio reaches at least 3, for 20° (R = 85m) the ratio becomes more than 4, and for 25° (R = 70m) more than 5. Generally it is to note that, for every 1° increase in degree of curve, an increase of about 1.4 accidents per million vehicle miles (Equation 26) can be expected regardless of the posted traffic warning devices, like recommended speeds, that decrease, as issue (3) reveals with increasing degree of curve.

To support the previous statements Figure 6 was developed, additionally. From this figure, related to database I [17, 18], observe that recommended speeds were rarely used on curves with degrees of curve less than about 5° of curve. Between 5° and 10° curves or curved sections were normally equipped with recommended speeds ranging from 50 to 35 mph, often combined
with arrow designations. And curves above 10° were mostly equipped with recommended speeds ranging from 35 to 20 mph, combined with arrow designations and chevrons. These observations coincide completely with those, stated under issue (3).

![Figure 6: Scatter Plot of 85th-Percentile Speed vs. Degree of Curve with Posted Recommended Speeds as Observed in the Field](image)

Thus, the research makes clear, that despite (sometimes) the stringent hardware installed at curved sites, accident rates are increasing with increasing degrees of curve (Figure 5b and issue 4), even though posted recommended speed limits are decreasing (Figure 4b and issue 2). These statements, obtained from investigations, conducted in the U.S.A., can be considered as "general", even though the magnitude of the accident rates certainly differ from country to country, depending on the road- and traffic conditions and especially on the performance level of traffic law enforcement.

The relationships of Figures 4 and 5, the considerations, explained under issues (1) to (4), the observations of Figure 6 and the T-test results of accident rates for changes in degree of curve classes between successive design elements (as discussed in Refs. 18, 20, 28 and 30) led to the recommended ranges for good and poor design practices and especially to the introduction of the fair design level of Table 1.

Furthermore, let us assume that the recommended speed should coincide, to a certain extent, with the design speed (defined in many geometric highway design standards as the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern). Then the speed differences, discussed under issue (3), suggest, at least for degrees of curve greater than about 10°, that the higher driving dynamic safety demand for the higher operating speeds (V85) exceeds definitely the driving dynamic safety supply, on which the design speeds are based, originally. Speed differences between 85th-percentile speed and design speed (recommended speed) of more than 12 mph (20 km/h) (poor design, Table 1) are not allowed (in Refs. 8, 10-13) or should not be allowed in any geometric design guideline.

But, because of a lack of financial resources in many countries, it does not appear possible to improve the large part of the existing low-volume, two-lane rural network, which corresponds according to Table 1 to fair design levels, by redesigning to good design levels. Therefore, the authors recommend in cases of "fair design" the proper use of traffic warning devices to
alleviate the problem of minor inconsistencies in geometric highway design, unless a serious safety problem does exist. But it should not be forgotten that the accident rates of changes in degree of curve up to 10° between successive design elements (fair design, Table 1) can reach values twice as high, as compared to those for changes in degree of curve up to 5° (good design, Table 1), according to Figure 5b and issue (4).

The potential usefulness of Figures 4 and 5 is evident. From these figures, the designer is able to roughly predict operating speeds (V85) and accident rates on curves or curved sections of two-lane, rural highways from beforehand knowledge of posted recommended speed limits or degree of curve. Furthermore he is able to determine the operating speed differences between successive design elements in order to assign those to the corresponding ranges for good, fair and poor design practices according to Table 1, as decision basis for further elaboration.

5. CONCLUSION

This study was undertaken to determine how TRAFFIC WARNING DEVICES (for example: posted recommended speed limits) do relate to operating speeds and accident rates on curves/curved sections of two-lane rural highways. A total of 322 road sections were considered for the study. For each of the curved sections, the following design parameters were collected: posted recommended speeds, degree of curve, curvature change rate (Kurvigkeit), length of curve, superelevation rate, gradient, sight distance, lane width, shoulder width, and average annual daily traffic.

It should be noted that degree of curve is an important U.S. design parameter. It is similar to the German design parameter, curvature change rate. In the case of a single design element (tangent or curve), degree of curve and curvature change rate are comparable.

Regression analyses indicated that posted recommended speed, degree of curve, or curvature change rate were able to explain most of the variation in operating speeds and accident rates on two-lane rural highways. The other parameters, with the exception of superelevation rate, contributed a little to the strength of the relationships.

In addition, the study revealed that with increasing degree of curve (1) the gap between operating speed, posted recommended speed, and design speed becomes wider and wider, and (2) accident rates show an increase, too despite the stringent recommended speeds, often combined with arrow designations and chevrons, installed at curved sites.

The potential usefulness of the relationships set forth in the study is that they provide the design engineer with regression models for (1) predicting operating speeds and accident rates on curved sections of two-lane rural highways, based on actual posted recommended speed plates, and (2) determining, from an actual knowledge of degree of curve of a certain roadway section, what recommended speed plates, if any, should be posted. Furthermore, the findings provide the engineer with an idea on where to draw the line between good, fair and poor designs, based on changes in operating speeds and accident rates. It should be noted that the studies have shown that the accident rate in the case of fair design is double that for good design, despite the posted traffic warning devices, but still a lot better than that for poor design.

The prediction models formulated in this study are only based on data from a limited geographic area and may only be appropriate for selected safety studies within that area (U.S.A., New York State). Some caution should be exercised in extrapolating the models to other areas or countries with differing laws, law enforcement, driving behavior, terrain, weather, and traffic control devices.

Acknowledgement

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25. "A Policy on Highway Types (Geometric)", Special Committee on Design Policies, AASHO, 1940.
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31. "A Policy on Highway Types (Geometric)", Special Committee on Design Policies, AASHO, 1940.
33. "A Policy on Highway Types (Geometric)", Special Committee on Design Policies, AASHO, 1940.
35. "A Policy on Highway Types (Geometric)", Special Committee on Design Policies, AASHO, 1940.
37. "A Policy on Highway Types (Geometric)", Special Committee on Design Policies, AASHO, 1940.
Traffic Safety Effects from Traffic Calming

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Traffic Safety Effects from Traffic Calming

by

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In Western-Germany federal governmental institutions are conducting experiments upon the effects of area-wide traffic calming measures in six cities. One of the issues being studied is traffic safety improvement. Up to now, studies have been finished for three of the towns under investigation. For the other towns preliminary results can be presented. The conclusion of the results is: Especially of occurrence heavy accident can be decreased by traffic calming. The number of severe personal injuries could be cut down to 60 per cent of the before-accident-figures. No comparable accident reduction could be experienced in the control areas without traffic calming. Those groups who earn the greatest benefit from traffic calming are pedestrians, children or two-wheel-drivers. Also a classification by types of accidents, by type of the street and junction will be presentation the paper.
TRAFFIC SAFETY EFFECTS FROM TRAFFIC CALMING

Introduction

In the Federal Republic of Germany, area-wide traffic calming measures are mean-while considered as an important means to improve the living conditions and the traffic situation in residential areas of cities. Area-wide traffic calming comprises several different kinds of measures which must be suited to each other in a conceivable and sensible way. The specialist literature on this subject is very extensive. These measures should also include the main roads of the respective areas. They must be accompanied by intensive relations work both with the motorists and the residents concerned and must be supervised appropriately.

The Federal Government is investigating the effects of these measures in a long scale experiment which includes analyses of the effects on the quality of traffic flow for cars, pedestrians, and bicycles, the development of urban structures, the effects on the living conditions of the residents, on social affairs, and on environmental impacts. The Ruhr-University, Bochum, has been commissioned to investigate traffic safety in such areas. The investigations are carried out in six cities (in parentheses: number of inhabitants of the whole city / of the investigated area): West Berlin (1,852,700/30,000 inhabitants), Mainz (188,200/28,000), Esslingen (87,300/11,100), Buxtehude (32,500/10,800), Ingolstadt (91,300/5,500), and Borgentreich (2,300/2,300). Final results from Berlin-Moabit [3] and Buxtehude [4] are already available. In the case of Esslingen, Ingolstadt and Borgentreich, there are only provisional results, since the periods scheduled for the investigation are not yet over (see table 1).

Methodology of the Investigation

The methodology of this accident analysis was developed and tested in a pilot study on a residential area in Berlin-Charlottenburg, where traffic calming measures were carried out between 1979 and 1982. From this area, final results are available as well [1], [2].

The accident analyses are based on the records of the respective police authorities, in which each accident is registered. These files include all accidents that are known to the police regarding their causes and consequences. For the analysis, each accident is evaluated according to 29 features concerning absolute accident rate (e.g. number of accidents, number of involved persons, number of specific consequences, accident costs) as well as relative accident parameters (e.g. accident rate, accident density). The accident occurrence before the beginning of the traffic calming measures and after the end of the measures is investigated. The length of these periods depends on the expected accident frequency in the investigated area. In detail, the investigation periods can be divided up as follows:
Furthermore, the accident occurrence in the investigated area is compared to an area which is similar regarding structure and traffic density, but in which no traffic calming measures have been carried out. This results in a before-and-after study with a control group. The data were analysed with the help of several statistical methods, like analyses of fourfold tables, log-linear approaches, a CHI\(^2\)-test, and Bayesian methods. As far as indices occur in the following tables and figures, they refer to the index 100, which is the mean of the accident rates in the period before the traffic calming measures.

### Subdivision of the investigated areas

#### Berlin-Charlottenburg

The investigated area in Charlottenburg is located near the Klausener Platz. Charlottenburg is a densely populated residential area with small business of the services and skilled trade. The buildings mainly date from the turn of the century and have been redeveloped in the course of activities to improve the living conditions in residential areas. In this area, a considerable number of streets was modified with various traffic calming measures, e.g. different levels of road surface, staggered lanes and regulation of parking vehicles. In order to come to a meaningful interpretation of accident rates, it is necessary to divide the investigated area up into parts of a similar structure. In Charlottenburg, the following divisions were made: CH1 - area in which street modifications ensure traffic calming, CH2 - area in which speed limit of 30 km/h was introduced, CH3 - in this area, south of the Knobelsdorffstraße, no modifications were made (passive traffic calming measures), CH4 - two adjacent roads with lower traffic volume, CH5 - the two neighbouring areas, CH6 - the arterials with high traffic volume which form the boundary of the investigated area, CH7 - the motorway approach Knobelsdorffstraße.

Berlin-Moabit was selected as the control group for economic reasons, because the accident data ascertained for the control area could be used as accident data for the period before the street modifications in Moabit.
Berlin-Moabit

The investigated area Berlin-Moabit includes the central part of Moabit which is located in the north of the district Berlin-Tiergarten. It is a densely populated residential area near the city centre with a central function for Moabit. The boundaries of the investigated area are water courses on three sides, industrial and commercial areas. Moreover, it is cut by two arterials with high traffic volume. For the investigation, Moabit was divided up into 8 parts: MO1 - with street modifications for traffic calming, MO2 - the area in which traffic calming measures were installed before the investigated period (Waldstraße), MO3 - the area south of the Turmstraße, MO4 - Two residential areas west of the Beusselstraße, MO5 - the Turmstraße as shopping street, MO6 - adjacent arterials with high traffic volume, MO7 - the adjacent arterials with low traffic volume, MO8 - three arterials west of the Beusselstraße. A part of the district Berlin-Wedding was chosen as control area.
Buxtehude

The municipal area of Buxtehude is cut into two parts of similar size by a railway running from the west to the east. The investigated area includes the northern half of the centre of Buxtehude with two shopping and service trade areas and residential areas with schools and sports grounds in the western and eastern directions. The investigated area was divided up into 6 parts: BU1 - the modified area with entire or partial street modification or road construction for traffic calming, BU2 - the 30km/h zone: traffic calming mainly by modifications of intersections, BU3 - collector roads with traffic calming measures from the north to the south, BU4 - the Bahnhofstr./Lange Str. as shopping street, BU5 - arterials within the investigated area, BU6 - arterials marking the boundary of the investigated area. The districts south of the railway in Buxtehude were chosen as control area.
Esslingen

The investigated area is situated in the municipal area. It is surrounded by the old part of the town in the north, by the city centre of Oberesslingen in the east and by the railway and the Neckar in the south. The area is characterized by its mixture of residential area, important industrial companies and commercial enterprises. A similar district of Nürtingen is the control area.

Ingolstadt

The investigated area includes almost the entire old part of the town with the exception of the fortress and the accompanying historical buildings, sports grounds and parking lots. The area is limited by the upper and lower moat in the west and the north, by the Paradeplatz in the east and by the streets Anatomiestraße, Am Münzbergtaal and by the Donau in the south. The enclosed structure of the medieval part of the town provides good preconditions for area-wide traffic calming measures. The old part of the town of Regensburg was chosen as control area.

Borgentreich

The investigated area includes the entire centre of Borgentreich. It is characterized by residential buildings, commercial enterprises and farms with large free spaces in private ownership in between. The centre is surrounded by a green belt which adjoins new housing estates in the west and the north.
Investigation results

Total number of accidents

The total number of accidents in the investigated and the control areas is presented in table 2 mainly to characterize the sample volume. Since absolute figures possess only limited validity, they have been related to the respective network lengths (accident density). Figures 4-6 demonstrate the accident densities for the areas from which final results are already available.

<table>
<thead>
<tr>
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<td>131</td>
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<td></td>
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</tr>
<tr>
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<td>438</td>
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<td></td>
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<td>1032</td>
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</tr>
<tr>
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<td>439</td>
<td>485</td>
<td>110</td>
</tr>
<tr>
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<td>85</td>
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<tr>
<td>control area</td>
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<td>486</td>
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<tr>
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<td>42</td>
<td>34</td>
<td>81</td>
</tr>
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</table>

Table 2. Total number of accidents

In Charlottenburg (fig. 4), the number of accidents has decreased significantly in the zone modified in the course of the traffic calming measures. A comparison of the residential areas (CH1 and MO8) shows that the reduction was three times higher in Charlottenburg than in Moabit. Moreover, the index values of the areas CH2 - CH4 are significantly lower than that of the residential areas (MO8) of the control area.

In the residential areas (MO1 - MO3) of the investigated area in Berlin-Moabit, the accident density has decreased by 10% - 32%, while there was an increase of 31% in the residential areas of the control area. This also shows a significant reduction of accidents in Moabit.

In Buxtehude, a decrease in accident numbers only occurred on the collector roads which were modified in the course of traffic calming measures. In the modified residential areas of the investigated area and the control area, accidents increased by 30%.
Fig. 4. Accident density in Berlin-Charlottenburg and Berlin-Moabit

Fig. 5. Accident density in Berlin-Moabit and Berlin-Wedding
Accident consequences

For the investigation, the most severe consequence of every accident was determined on the basis of the police reports. The accidents were divided up as follows:

- slight damages
- severe damages
- slightly injured persons
- severely injured persons
- fatalities.

In table 3, the index values for damage only accidents are presented as well as for the number of persons injured or killed in an accident. The table contains only the results from those parts of the investigated area in which traffic calming measures had been carried out in comparison with the results from the respective residential areas of the control area.

The index values marked (*) in this and further tables indicate that in these cases the accident feature occurred less than 5 times in the period before and after the modifications.
In all areas with traffic calming measures, the number of persons injured in an accident decreased significantly. With the exception of the control area of Esslingen, there has also been a reduction of slightly injured persons in the residential areas of the control areas, but these decreases are far less significant. With the exception of Borgentreich, the other 5 areas in which traffic calming measures were carried out have a mean index value for slightly injured persons of I=51% opposed to I=89% in the control areas. In the modified areas, the number of severely injured persons could on average be decreased by one third of the value before the modification (mean index value I=67%).

Regarding the damage only accidents, no general development within the investigated towns can be detected. For Berlin-Charlottenburg, table 2 already showed a decrease in the total amount of accidents. The reduction of slight and severe damage only accidents (decrease by 28% each) reflects this tendency. In Berlin-Moabit and Buxtehude, a development from severe to slight damages can be observed, whereas in Esslingen and Ingolstadt, severe damages have increased significantly in the period after the modifications. The index value of $I=700\%$ for Borgentreich includes an increase in accidents with severe damages from 1 before to 7 after the modifications. The results from Esslingen and Ingolstadt should be interpreted cautiously, because in both cases, the last year of the investigated period after the street modifications has not been evaluated yet. Thus, certain accident features can still change significantly until the final results are available. However, it has already become obvious that far less persons were injured in accidents after the introduction of traffic calming measures.

### Table 3. Index values for damage only accidents and for the number of injured persons

<table>
<thead>
<tr>
<th></th>
<th>slight damages</th>
<th>severe damages</th>
<th>slightly injured persons</th>
<th>severely injured or killed persons</th>
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<td></td>
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<td>72</td>
<td>72</td>
<td>33</td>
<td>* 50</td>
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<tr>
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<td>91</td>
<td>91</td>
<td>69</td>
<td>118</td>
</tr>
<tr>
<td>Moabit</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>127</td>
<td>43</td>
<td>44</td>
<td>27</td>
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<tr>
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<td>59</td>
<td>83</td>
<td>52</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
</tr>
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<td>41</td>
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<tr>
<td>control area</td>
<td>173</td>
<td>32</td>
<td>47</td>
<td>100</td>
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<tr>
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<td></td>
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<td></td>
<td></td>
</tr>
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<td>111</td>
<td>152</td>
<td>89</td>
<td>56</td>
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<td>control area</td>
<td>90</td>
<td>146</td>
<td>187</td>
<td>-</td>
</tr>
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<td>Ingolstadt</td>
<td></td>
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<td>121</td>
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<td>70</td>
<td>98</td>
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<td>48</td>
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<td></td>
<td></td>
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</tr>
<tr>
<td>investigated area</td>
<td>79</td>
<td>* 700</td>
<td>22</td>
<td>60</td>
</tr>
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</table>

In all areas with traffic calming measures, the number of persons injured in an accident decreased significantly. With the exception of the control area of Esslingen, there has also been a reduction of slightly injured persons in the residential areas of the control areas, but these decreases are far less significant. With the exception of Borgentreich, the other 5 areas in which traffic calming measures were carried out have a mean index value for slightly injured persons of $I=51\%$ opposed to $I=89\%$ in the control areas. In the modified areas, the number of severely injured persons could on average be decreased by one third of the value before the modification (mean index value $I=67\%$).
Fig. 7. Density of accident costs in Berlin-Charlottenburg and Berlin-Moabit

Fig. 8. Density of accident costs in Berlin-Moabit and Berlin-Wedding
In the three towns for which final results are already available, the accident costs in the investigated areas have been ascertained on the basis of the evaluation method for damage only accidents and accidents with injured persons on urban roads which the RAS-W (1986) [8] is also based on. These accident costs are losses for the national economy. Their registration and evaluation is of particular importance, because they include number and severity of accidents. On the other hand, the accident costs are related to the respective network lengths (density of accident costs).

These figures show that the density of accident costs could be reduced significantly in those areas in Charlottenburg (fig.7), Moabit (fig.8) and Buxtehude (fig.9) in which traffic calming measures were carried out. The residential areas of the control areas do not show comparable reductions within the investigated periods.

**Involved persons**

One of the aims of traffic calming measures is to improve traffic safety for the weaker road users. Therefore, it was important to find out which group of road users was particularly often involved in accidents before and after the measures were carried out.

Table 4 lists up the index values for the accident involvement of different groups of road users in the parts of the investigated areas in which traffic calming measures were installed and in the residential areas of the control areas. Bortentreich has been left out in this table, because the comparison of the situation before and after the modifications resulted in insignificant differences: passenger cars 57/50, cyclists 1/3, motorcyclists 6/0, pedestrians 1/1.
Table 4. Index values for the accident involvement of different groups of road users.

The most important result of table 4 is the significant reduction of motorcyclists and pedestrians who were involved in accidents in all five investigated areas after traffic calming measures had been carried out. With the exception of the motorcyclists in the control area of Charlottenburg (increase from 9 to 17) and the pedestrians in the control area of Buxtehude (increase from 2 to 7), positive developments could be observed in the respective control areas as well. These improvements, however, are far less significant than in the areas with traffic calming measures. The mean index value for motorcyclists is \( I = 32\% \) in the investigated areas and \( I = 76\% \) in the control areas. The mean index value for pedestrians is \( I = 48\% \) (investigated areas) and \( I = 126\% \) (control areas).

The accident involvement of cyclists has increased from 15 to 16 in Buxtehude and from 9 to 18 in Esslingen. In the other areas, the safety of cyclists has improved considerably. The same is true for the involvement of children in accidents. An increase in accidents with children was only observed in Esslingen (from 9 to 12).

**Types of accidents**

Furthermore, it was investigated whether traffic calming measures had different effects on the individual types of accidents. The 13 accident types collected in the statistics are here summarized in 6 groups:

- **Type 1**: Accidents during the driving process
- **Type 2**: Accidents due to turning and crossing maneuvers
- **Type 3**: Accidents in connection with parking
- **Type 4**: Accidents with pedestrians crossing the road
- **Type 5**: Accidents in parallel traffic with road users of the same direction
- **Type 6**: Accidents in parallel traffic with road users of the opposite direction

It turned out that traffic calming measures mainly reduced the number of accidents at intersections (type 2). Nevertheless, the share of these accidents in the total
amount of accidents is still big. Considerable improvements regarding accident type 4 could be achieved in all five investigated areas. The insignificant involvement of pedestrians confirms this statement. With the exception of Moabit, rear end collisions (type 5) are also on the decrease. On the other hand, accidents in parallel traffic with road users of the opposite direction have obviously increased in the modified areas. These increases often result from the structural changes (bottlenecks, staggered lanes) built in the course of traffic calming measures. This type of accident, however, mostly results in slight damages.

<table>
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<td>* 400</td>
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<td>* 100</td>
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Table 5. Index values for different types of accidents

Summary

In Charlottenburg, the total number of accidents has decreased significantly in the course of the traffic calming measures. Street modifications have proven to be more effective than speed limits. Negative effects due to the shift of traffic to other areas were not ascertained.

In Berlin-Moabit, traffic safety has been improved in the area in which street modifications were installed (10% reduction of accidents, 60% reduction of accidents with injured persons, halved accident costs). Children and two wheel traffic benefitted particularly from the traffic calming measures (73% decrease in injured children, 53% decrease in accidents with cyclists and motorcyclists). This is confirmed by a comparison with the control area Berlin-Wedding. The traffic calming measures have had no negative effect on the arterials as far as traffic safety is concerned.

Within the modified area of Buxtehude (BU1, BU3), the average accidents are rather slight. Although the total number of accidents has increased by 17%, there has been a shift from severe damages (-44%) to slight damages (+48%). The number of persons injured in an accident was decreased from 41 to 16 (59% decrease in slightly injured persons, 67% decrease in severely injured persons). This results in a reduction of accident costs to two thirds of the value before the
modifications. The involvement of motorcycles was reduced from 18 to 3 and the involvement of pedestrians from 7 to 1 person.
The result of the accident analysis in the investigated areas Berlin-Charlottenburg, Berlin-Moabit and Buxtehude show that traffic calming measures have significantly improved traffic safety. For the other investigated towns, final statements can be made when the entire period of investigation has been evaluated. These results are expected to be available in 1991.

References


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Statistical Distribution of Speeds on German Motorways

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ABSTRACT

Figures on the overall mileage under defined conditions are more or less available for the motorway networks of most countries. However, the proportion of mileage performed in different speed classes is usually not known; this is regrettable because speed has a very significant impact on traffic safety and on air pollution. Therefore, the objective of the present paper is to establish the speed distribution of all passenger cars for a given year (1987) on the entire network of West-German motorways. More precisely, speed distribution here is understood to be the proportion of mileage in the speed classes from 0 to 60 km/h, from 60 to 70 km/h and so on in classes of 10 km/h width up to the class from 170 to 180 km/h as well as an overflow class above 180 km/h.

The method consisted of three steps. In the first step the traffic volumes were determined for each direction and each section of the German motorway network as well as for each hour of the year. To this end, data of traffic counts were used.

In the second step various speed-flow relationships were developed from specific microscopic measurements. These relationships were linked to the volumes in order to yield mean speeds for each part of the network and each hour of the year.

In the third step, the speeds were distributed about their mean values and finally the proportions in each speed class were aggregated in order to obtain values that are valid for the overall network.

The different steps were then combined to yield the overall speed distribution. The results show that 15% of the passenger car mileage in 1987 was achieved while the speeds were above 145 km/h and that 31% were above 130 km/h. The mean speed turned out to be 117,2 km/h.

Since these figures refer to a complete year, the seasonal variations that are inherent in the speed distribution were additionally investigated. Detailed data were available from five permanent measurement stations. A statistical model was developed from which some temporal components of variation could be extracted.
1. INTRODUCTION

Figures on the overall mileage under defined conditions are more or less available for the motorway networks of most countries. However, the proportion of mileage performed in different speed classes is usually not known; this is regrettable because speed has a very significant impact on traffic safety and on air pollution. Therefore, the main objective of the present paper is to establish the speed distribution of all passenger cars for a given year (1987) on the entire network of West-German motorways. More precisely, speed distribution here is understood to be the proportion of mileage in speed classes of, e.g., 10 km/h width.

The method consists of three steps. In the first step (c.f. Chapter 2) the traffic volumes were determined for each direction and each section of the German motorway network as well as for every hour of the year. To this end, data of traffic counts are used.

In the second step (c.f. Chapter 3) various speed-flow relationships were developed from specific microscopic measurements. These relationships are linked to the volumes in order to yield mean speeds for each part of the network and every hour of the year.

In the third step (c.f. Chapter 4), the speeds are distributed about their mean values and finally the proportions in each speed class are aggregated in order to obtain values that are valid for the overall network.

Chapter 5 contains the theoretical model for combining the results from the different steps. The overall results for the speed distribution referring to the entire year 1987 are given in Chapter 6. More details are given in [10].

The first version of the a.m. procedure was developed in [1], using data from the year 1982. The method was essentially applied in the large-scale exhaust gas test ("Abgas-Großversuch") [2] with data from 1985.

In Chapter 7 of the present paper, finally, the seasonal variations that are inherent in the speed distribution are additionally investigated. Detailed data were available from some automatic long-term counting stations. A statistical model is developed which allows some temporal components of variation to be extracted; this includes trends over several years.

1) The reader should be aware of the fact that in West Germany there is no general speed limit on the motorway network. However, a maximum speed of 130 km/h is recommended.
2. DETERMINATION OF TRAFFIC VOLUMES

In the Federal Republic of Germany, nation-wide general traffic counts ("Bundesverkehrszählungen") are undertaken every five years (the last one in 1985) [6], based on "manual" counting data for all motorway segments from which AADT values are estimated. Apart from that, more than 300 automatic long-term counting stations are installed within the West-German motorway (BAB) network for continuous traffic volume recordings on an hourly basis. Both data sources were used to compute the passenger car volumes \( q(h,i,r) \) for every section \( i (i = 1, \ldots, 1668) \) of the BAB network, for each direction \( r (r = 1, 2) \) and every hour \( h (h = 1, \ldots, 8760) \) of 1987.

The determination of \( q(h,i,r) \) was carried out in two phases. To begin with each section \( i \) was assigned a section \( j = j(i) \) with an automatic long-term counter equipped with an analyser to distinguish between passenger cars and trucks [4]. Secondly, the relation

\[
q(h,i,r) = q(h,j,r) \cdot \frac{\text{AADT}_{pc,i}}{\text{AADT}_{pc,j}}
\]

was established where \( \text{AADT}_{pc,i} \) and \( \text{AADT}_{pc,j} \) respectively denotes the AADT value of passenger cars for section \( i \) and \( j \) respectively in 1985.

The associated percentage of heavy freight vehicles in the overall motor vehicle traffic (truck percentage) for section \( i \) in each direction and for every hour was adopted from section \( j \).

This procedure was applied to 1668 BAB segments of a total length of 7756 km. The total length of the BAB network was 8198 km on Jan. 1, 1985 and 8437 km on Jan. 1, 1987.

3. V-Q RELATIONSHIPS FOR PASSENGER CARS

In order to determine the required speed-flow relationships (v-q relationships) microscopic data from regular empirical studies of the Federal Highway Research Institute (BAST) [5] were used. The relationships were developed separately for the following classes of truck percentages:

- on two-lane unidirectional roadways:
  - < 5 \%, 5 to < 15 \%, 15 to < 25 \%, \geq 25 \%;

- on three-lane unidirectional roadways:
  - < 15 \% and \geq 15 \%.

For BAB segments with a speed limit, a classification by truck percentages was not used since the available data did not yield any effects on passenger car speeds worth mentioning.

The distinction according to truck percentages also led to the establishment of different capacities \( q_{max} \) for the various v-q relationships. At low truck percentages, the capacity (i.e., here the maximum possible passenger car volume) is higher than at high truck percentages. When capacity was reached, a mean passenger car speed

\( 2^) \) AADT = annual average daily traffic

VTI RAPPORT 363A
speed\(^3\) of 75 km/h was uniformly assumed (with the exception of work sites). Where demands exceeded the capacity, linear interpolation of mean speeds beyond capacity was carried out. Values below 20 km/h however were raised to 20 km/h.

The v-q relationships described below are based on data collected in daylight, in dry weather and outside of the winter season. Effects on passenger car speeds, e.g. of nighttime conditions or unfavourable weather, are not accounted for by any special v-q relationships, but are comprehensively adjusted for in the calculation model later on. (cf. Chapter 5).

The v-q relationships for two-lane unidirectional roadways are represented in Fig. 1. They are based on data from 13 counting stations. The v-q relationships for three-lane unidirectional roadways are given in Fig. 2. In this case the data are obtained from 7 counting stations.

**Existing speed limits**

The v-q relationships distinguish between the permanent speed limits of 80 km/h and 100 km/h and the temporary limits for auxiliary lanes at work sites (cf. Fig. 3).

Concerning the permanent speed limits, up-to-date data for a two-lane section with a speed limit of 100 km/h were available. The corresponding v-q relationship has been adjusted by means of the data measured in the large-scale exhaust gas test ("Abgas-Großversuch") [2]. For sections with a speed limit of "80 km/h", up-to-date data were not available. Therefore, a model compatible with that for the limit of "100 km/h" was developed. In addition, the model was also adjusted for compatibility with the data measured in the large-scale exhaust gas test.

The v-q relationships for auxiliary lanes at work sites were modelled on the basis of studies undertaken by BECKER et al. [7]. Based on v-q relationships available for various speed limits (60 km/h and 80 km/h) and various forms of traffic control (4+0 and 3+1), it was attempted to derive a "mean" v-q relationship universally applicable to all work sites.

**Steepness of grades**

Based on the v-q relationships determined for level BAB segments which were also applied to all downgrades and upgrades of less than 2 %, the steeper upgrade sections were taken into account by constant reduction of the v-q relationship. Both for two-lane and three-lane unidirectional roadways the reductions below, given in [8], were employed:

\(^3\) In Chapters 3 through 6 speed distributions are understood to be spot speed distributions, i.e. they refer to points at the roadway, not to points of time. The corresponding mean spot speeds are sometimes called time mean speeds in the literature.
- upgrade $< 2\% : \pm 0 \text{ km/h}$,
- upgrade $2 - 3\% : - 11 \text{ km/h}$,
- upgrade $3 - 4\% : - 18 \text{ km/h}$,
- upgrade $4 - 5\% : - 25 \text{ km/h}$,
- upgrade $> 5\% : - 30 \text{ km/h}$.

**Figure 1:** v-q-relationships for two-lane unidirectional roadways (without speed limit)

**Figure 2:** v-q-relationships for three-lane unidirectional roadways (without speed limit)
Figure 3: v-q-relationships for unidirectional roadways with speed limits

4. PASSENGER CAR SPOT SPEED DISTRIBUTION

Comprehensive data material for two-lane unidirectional roadways was available from continuous automatic speed measurements at 10 BAB unidirectional cross sections (cf. Chapter 7). For each of these cross sections a data record comprising mean passenger car speeds and passenger car speed distributions (in the speed classes 0-60 km/h, 60-70 km/h, etc. by classes of 10 km/h width, 170-180 km/h, and the overflow class above 180 km/h) in addition to passenger car and truck volumes was available for every hour of 1987. The data were processed as follows:

Each of these data records was grouped into one of 96 categories resulting from the combination of 24 categories of mean hourly passenger car speeds and 4 categories of hourly truck percentages. The latter were defined as stated in Chapter 3 (< 5%, 5 to < 15 %, 15 to < 25% and ≥ 25% ). The categories of mean passenger car speeds were uniformly of a width of 5 km/h with category 1 covering 20 to 25 km/h and category 24 finally 135 km/h to 140 km/h. The data records in each of these 96 categories were then aggregated to yield the speed distribution (with the classes above) valid for the category in question. Examples of these distributions are shown in Fig. 4.
Figure 4: Hourly distributions of passenger car speeds for three categories of mean speeds and one category of truck percentage (two-lane unidirectional roadways)

At high speeds, each of these speed distributions was cut off for sections with speed limits by determining the maximum of 120 km/h and that of 1.7 times the respective mean speed and adding the portions exceeding these maxima to the highest remaining class (below the maximum). Distinguishing between the 4 categories of truck percentages did not appear to be useful in the case of speed limits so that the 24 categories of mean passenger car speeds only had to taken into consideration.

Normal distribution was assumed for passenger car speeds on three-lane unidirectional roadways. For the complete determination of the speed distributions, therefore, it was only necessary to determine the coefficients of variation in addition to the mean speeds given by the v-q relationships. Empirical studies revealed that these coefficients of variation decrease from 0.20 to 0.16 at passenger car volumes from 0 to 2500 veh./h, reaching a constant value of 0.16 from 2500 veh./h. A constant variation coefficient of 0.16 was assumed without exception for sections with speed limits.

The speed distributions applying to level sections were also assumed to apply to upgrade sections as the effect of upgrades is taken to be accounted for in the constant reductions of the v-q relationships.
5. CALCULATION MODEL

5.1 Basic theoretical model

Let AM\textsubscript{i,r,p}(v) be the annual mileage of all passenger cars at speeds below v in the direction of r on the \textit{i}\textsuperscript{th} BAB segments if the truck percentage is in the \textit{p}\textsuperscript{th} category (l=1,...,4). Let the length of this section be l\textsubscript{i}. Furthermore let q=h,i,r) denote the passenger car volume on that section within the hour h in travel direction r, and \(F\textsubscript{q,p}(v)\) the proportion\textsuperscript{1}) of passenger cars travelling at volume \textit{q} (with truck percentages in the \textit{p}\textsuperscript{th} category) and speeds below \textit{v}. Let us determine AM\textsubscript{i,r,p}(v) first.

The mileage of all passenger cars on the \textit{i}\textsuperscript{th} BAB segment in direction \textit{r} and hour \textit{h} now equates \(q(h,i,r)l\textsubscript{i}\). Speeds below \textit{v} yield a contribution to this value amounting to \(q(h,i,r)l\textsubscript{i}\cdot F\textsubscript{q,p}(v)\). If \(H(p)\) denotes the total of hours of a year (1987) with a truck percentage in the \textit{p}\textsuperscript{th} category, we obtain by summation for these 8760 hours of the year:

\[
AM_{i,r,p}(v) = \sum_{h=1}^{8760} q(h,i,r)l\textsubscript{i}\cdot F\textsubscript{q,p}(v)\cdot 1\textsubscript{H}(h).
\]

(1) AM\textsubscript{i,r,p}(v)= \(\sum_{h=1}^{8760} q(h,i,r)l\textsubscript{i}\cdot F\textsubscript{q,p}(v)\cdot 1\textsubscript{H}(h).\)

(2) AM\textsubscript{i,r,p}(v)= \(l\textsubscript{i}\cdot \sum_{q=0}^{q_0} N_{i,r,p}(q)\cdot F\textsubscript{q_0,p}(v),\)

where the sum extends over all lower limits \(q_0\) of the classes of hourly passenger car volumes starting with \(q_0=0\). \(q_0=q_0+\Delta q/2\) is the mean of a class in each case. \(N_{i,r,p}(q_0)\) denotes the number of hours in a year with passenger car volumes between \(q_0\) and \(q_0+\Delta q\) on the \(i\textsuperscript{th}\) BAB segment in direction \textit{r} with a truck percentage in the \textit{p}\textsuperscript{th} category.

Summing up the BAB segments (index \(i\)), the directions of travel (index \(r\)) and the categories of truck percentages (index \(p\)) yields the equation below to determine AM\textsubscript{v}, which is the annual passenger car mileage (in 1987) at speeds below \textit{v} on all the BAB segments evaluated:

\[
AM(v) = \sum_{i} \left(1\textsubscript{i} \cdot \sum_{r} \left(\sum_{p} (q_0) N_{i,r,p}(q_0)\cdot F\textsubscript{q_0,p}(v)\right)\right).
\]

(3) AM\textsubscript{v}=

In order to obtain the proportion of passenger car speeds below \textit{v}, weighted by the annual mileage, \(F(v)\), equation (3) must be divided by the annual passenger car mileage given by AM(\textsubscript{\infty}) (note: \(F\textsubscript{q_0,p}(\infty)\) must be equal to 1):

\[
F_{q_0,p}(v) = \text{to be determined as a proportion of spot speeds, not as a proportion of instantaneous speeds (cf. [1])}.
\]
(4) \( F(v) = \frac{AM(v)}{AM(\infty)} \).

The mean passenger car speed, \( \bar{v} \), weighted by the annual mileage, finally is obtained by integrating the speed \( v \) over the distribution given by \( F(v) \):

\[
(5) \quad \bar{v} = \frac{\int_0^\infty v \cdot dF(v)}{AM(\infty)} = \frac{\sum_i \sum_r p \cdot q_0 \cdot \left( \sum_i q_0 \cdot N_i \cdot r \cdot p(q_0) \cdot \int_0^\infty v \cdot dF_{q_0,p}(v) \right)}{AM(\infty)}
\]

5.2 Boundary conditions

The speed distribution \( F(v) \) weighted by the annual mileage depends, among other factors, on the presence of upgrades, speed limits and work sites.

The percentage lengths of upgrade sections on BAB unidirectional roadways were determined for the upgrade classes < 2%, 2-3%, 3-4%, 4-5% and ≥ 5%. The percentage lengths of sections with speed limits (100 km/h and 80 km/h) were calculated for each German state and the types of cross sections under consideration (two-lane and three-lane unidirectional roadways). The percentage lengths of work sites finally were assumed to be a uniform 4% for all states and cross sections since details did not exist.

At first, the German state and cross section type was determined for each BAB segment and direction of travel. This was followed by calculating the probability of a specific upgrade class on the one hand and the existence of a speed limit or a work site on the other hand as a product of the respective individual probabilities (= percentage lengths).

At a given traffic volume, the mean passenger car speed in all cases was expressed by the minimum resulting values of the \( v \cdot q \) relationships under the respective boundary conditions.

In accordance with (5) the value of 121.0 km/h is finally obtained for \( v \). This value already applies to the entire BAB network, although factors, such as heavy rain, fog, incidents (e.g., congestion due to accidents), nighttime and winter conditions are not yet taken into account. It is estimated that due to these factors the mean passenger car speed is reduced by about 3.8 km/h to 117.2 km/h. The component parts of this reduction are explained in [10]. Assuming individual speeds to be reduced by a constant factor, the final distribution function \( G(v) \) obtained by modifying \( F(v) \) is then the following:

\[
G(v) = F(v \cdot 121/117.2).
\]
6. RESULTS

The model calculations yield the relative annual passenger car mileage on the BAB network grouped by speed classes which are listed in Table 1. In this passenger car speed distribution, speed reduction factors, such as the ones below, are taken into account:

- speed limits, including those at work sites
- upgrades
- high traffic densities
- incidents caused by adverse weather conditions and
- congestion caused by accidents.

<table>
<thead>
<tr>
<th>Speed class (km/h)</th>
<th>Proportion of total annual passenger car mileage (in %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 60</td>
<td>1.8</td>
</tr>
<tr>
<td>&lt; 70</td>
<td>3.6</td>
</tr>
<tr>
<td>&lt; 80</td>
<td>7.5</td>
</tr>
<tr>
<td>&lt; 90</td>
<td>14.2</td>
</tr>
<tr>
<td>&lt; 100</td>
<td>24.5</td>
</tr>
<tr>
<td>&lt; 110</td>
<td>38.2</td>
</tr>
<tr>
<td>&lt; 120</td>
<td>53.9</td>
</tr>
<tr>
<td>&lt; 130</td>
<td>68.7</td>
</tr>
<tr>
<td>&lt; 140</td>
<td>80.7</td>
</tr>
<tr>
<td>&lt; 150</td>
<td>89.2</td>
</tr>
<tr>
<td>&lt; 160</td>
<td>94.5</td>
</tr>
<tr>
<td>&lt; 170</td>
<td>97.3</td>
</tr>
<tr>
<td>&lt; 180</td>
<td>99.3</td>
</tr>
<tr>
<td>&lt; ∞</td>
<td>100.0</td>
</tr>
</tbody>
</table>

We thus obtain the following passenger car speed distribution characteristics with respect to the annual mileage:

- mean speed, \( \bar{V} = 117.2 \text{ km/h} \),
- 85th percentile speed\(^5\), \( V_{85} = 145.1 \text{ km/h} \),
- proportion of passenger cars with speeds exceeding 130 km/h, \( P(v>130) = 31.3 \% \).

7. SEASONAL FLUCTUATIONS AND MULTIYEAR TRENDS

For several years, BASt has operated automatic long-term counting stations at 10 unidirectional cross sections of roadways of West-German motorways measuring not only traffic volumes but also speed data. In the following, the evaluation of the speed data collected from 1978 to 1989 is presented. In the evaluation, particular emphasis was placed on the analysis of the effects of temporal factors (year, month, weekday) on the speed distribution characteristics. Further information is given in [9].

\(^5\) The 85th percentile speed is the speed value which is exceeded in 15 per cent of all cases.
The speed measurements for each hour and lane are grouped into up to 20 classes. Based on the data recorded per hour and lane, the following factors were determined at first:

- vehicle and truck volumes
- number of motor vehicles in the following speed classes: less than 110 km/h, 110-130 km/h, 130-150 km/h and greater than 150 km/h
- space mean speed of motor vehicles
- traffic density

Based on the hourly data, monthly data were aggregated to provide the basis of the statistical analysis. The aggregations were carried out separately for each counting station and lane and for defined weekday classes: "Monday to Friday", "Saturday", "Sunday and holidays".

The following parameters were determined for each year, month, counting station, lane and weekday class:

\[ \bar{v};_i \] = mean value of hourly motor vehicle space mean speeds

\[ v^i \] = percentage of traffic in speed class i (cf. Table 2)

\[ k \] = mean value of hourly traffic densities

\[ p \] = mean value of hourly truck percentages

Table 2: Speed class boundaries

<table>
<thead>
<tr>
<th>Speed class No.</th>
<th>1 [km/h]</th>
<th>2 [km/h]</th>
<th>3 [km/h]</th>
<th>4 [km/h]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed class</td>
<td>110 - 130</td>
<td>130 - 150</td>
<td>&gt; 150</td>
<td></td>
</tr>
</tbody>
</table>

7.1 Statistical model

We proceed from the assumption that the target values \( \bar{v} \) (mean monthly speed) and \( v^i \) (monthly percentage of motor vehicles in speed class i) calculated from the data material are subject to the effects of time, space and traffic conditions. As an example, each value of \( \bar{v} \) or \( v^i \) value will be subject, among other influencing factors, to the effects of the year, month, weekday class, counting station, lane, the associated traffic density and truck percentage. In order to be able to separately assess and quantify these simultaneously occurring and overlapping effects, a suitable statistical model is an essential requirement [11].

The obvious solution here was the use of a variance analysis model comprising the main effects of the temporal and spacial factors of influence already mentioned and, additionally, the traffic effects as covariate.

The underlying model approach takes the following form:

\[
\bar{v} = \mu + \sigma_j + \beta_m + \gamma_t + \delta_z(i) + \varepsilon_{jm}(z(i)) + a_1\cdot(k_1 - \bar{k}_1) + a_2\cdot(k_2 - \bar{k}_2) + b_1\cdot(p_1 - \bar{p}_1) + b_2\cdot(p_2 - \bar{p}_2) + u.
\]
Where:

\[\begin{align*}
\mu & : \text{constant} \\
\alpha_j & : \text{main effect of year } j \ (j = 1978, \ldots, 1989) \\
\beta_m & : \text{main effect of month } m \ (m = 1, \ldots, 12) \\
\gamma_t & : \text{main effect of weekday class } t \ (t = 1: \text{Monday to Friday}, t = 2: \text{Saturday}, t = 3: \text{Sunday and holidays}) \\
\delta_{z(i)} & : \text{main effect of traffic lane } i \text{ at counting station } z \\
\epsilon_{jm}(z(i)) & : \text{main effect of counting equipment on lane } i \text{ at counting station } z, \text{ if the equipment had been changed in month } m \text{ of year } j \\
k_i & : \begin{cases} k \text{ (traffic density) on lane } i \\ 0, \text{ otherwise} \end{cases} \\
p_i & : \begin{cases} p \text{ (truck percentage) on lane } i \\ 0, \text{ otherwise} \end{cases} \\
\bar{k}_i \text{ or } \bar{p}_i & : \text{mean of covariates } k_i \text{ or } p_i \\
a_1, b_1 & : \text{regression coefficients} \\
u & : \text{residual random component (0 mean value)}
\end{align*}\]

The influencing factor \(\epsilon\) takes account of any shifts in measurements caused by the re-equipping of counting stations with modern instruments during the years 1986-87.

The model approach (6) here has been formulated to only consider \(\bar{\nu}\) as dependent variable. A similar approach, however, is employed for considering the variables \(\nu^1\) instead of \(\nu\).

The statistical estimate of the model parameters was carried out based on the least squares method. The main effect of the factors and of the covariates were adjusted simultaneously. This is to say that the factors and covariates were not assumed to be hierarchically structured. It finally also has to be pointed out that interaction effects were not considered because of the limited storage capacity of the computer and software (SPSS) used. However, even without these restrictions, the interaction effects of the experimental design would have been adjusted after the main effects of factors and covariates because the design was unbalanced due to some missing data. Thus the values of the primarily interesting model parameters would have remained the same.

In order to avoid linear dependencies that would have destroyed the statistical estimability of the main effects, constraints were so formulated as to equate the constant \(\mu\) in (6) with the mean of all observations of the respective dependent variable, i.e. the so-called "grand mean".

Having adjusted model (6) in the form described, it was possible to determine the values of each dependent variable which are due exclusively to the effect of any of the five factors involved. This only requires setting the other four factors and the residual component to 0 and equating the covariates with their respective means so that, e.g., for the \(\bar{\nu}\) values due only to the effect of the year and corrected by eliminating all other effects, the following is obtained:

\[\bar{\nu}^1 \text{ denotes statistical estimates.}\]
\[ \overline{v}(j) = \hat{\mu} + \hat{\alpha}_j \quad (j = 1978, \ldots, 1989). \]

The values attributable to the effects of a month or a weekday class are obtained in analogous manner.

An analogous procedure also applies to the variables \( v^i \) in place of \( \overline{v} \).

### 7.2 Results

The adjustment of model (6) to the data proved to be satisfactory. For each degree of freedom used up by the model, more than 70 observations were available. The effects of individual model components on dependent variables were in general highly significant - with the singular exception concerning the covariate \( p_2 \) and the variable \( v^4 \).

The main effects of temporal factors and the "grand means" are listed in Table 3. In each case, the values listed have been adjusted for all other effects.

The values in Table 3 show primarily the relative differences between the levels of the different factors. However, in contrast to the results of Chapter 6 there are no claims for the absolute values of Table 3 to be valid for the entire BAB network.

Table 3 shows in particular that the mean speed on motorways

- increased by about 10 km/h between 1978 and 1989 with an increase of about 5 km/h from 1986 to 1989 alone. Reductions compared with the year before were found in 1980 and 1981 (presumably due to rising fuel prices) and also in 1985 (the year when the large-scale exhaust gas test took place) and in 1987.

- was clearly lower in the winter months December-February than in the remaining months of the year. The maximum difference found between two months (January and April) was about 10 km/h.
**Table 3:** Grand means and main effects of the factors year, month, and weekday class

<table>
<thead>
<tr>
<th>Portions</th>
<th>( \bar{V} ) [km/h]</th>
<th>( \leq 110 ) km/h</th>
<th>110 to 130 km/h</th>
<th>130 to 150 km/h</th>
<th>( &gt; 150 ) km/h</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Grand mean</strong></td>
<td></td>
<td>116.67</td>
<td>37.72</td>
<td>31.11</td>
<td>20.98</td>
</tr>
<tr>
<td><strong>Year:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1978</td>
<td>-4.59</td>
<td>-2.36</td>
<td>4.00</td>
<td>1.17</td>
<td>-2.60</td>
</tr>
<tr>
<td>1979</td>
<td>-4.05</td>
<td>1.58</td>
<td>1.65</td>
<td>-0.88</td>
<td>-2.21</td>
</tr>
<tr>
<td>1980</td>
<td>-4.26</td>
<td>5.52</td>
<td>0.25</td>
<td>-3.08</td>
<td>-2.55</td>
</tr>
<tr>
<td>1981</td>
<td>-4.59</td>
<td>5.41</td>
<td>1.15</td>
<td>-3.56</td>
<td>-2.84</td>
</tr>
<tr>
<td>1982</td>
<td>-3.53</td>
<td>4.17</td>
<td>1.19</td>
<td>-2.87</td>
<td>-2.38</td>
</tr>
<tr>
<td>1983</td>
<td>-2.58</td>
<td>2.07</td>
<td>1.57</td>
<td>-1.69</td>
<td>-1.79</td>
</tr>
<tr>
<td>1984</td>
<td>-2.13</td>
<td>1.96</td>
<td>0.05</td>
<td>-0.85</td>
<td>-0.93</td>
</tr>
<tr>
<td>1985</td>
<td>-2.50</td>
<td>3.60</td>
<td>0.80</td>
<td>-2.42</td>
<td>-1.83</td>
</tr>
<tr>
<td>1986</td>
<td>0.32</td>
<td>-1.01</td>
<td>0.90</td>
<td>0.25</td>
<td>-0.20</td>
</tr>
<tr>
<td>1987</td>
<td>-1.67</td>
<td>3.95</td>
<td>-2.95</td>
<td>-1.13</td>
<td>0.07</td>
</tr>
<tr>
<td>1988</td>
<td>3.51</td>
<td>-3.39</td>
<td>-0.29</td>
<td>1.83</td>
<td>1.75</td>
</tr>
<tr>
<td>1989</td>
<td>5.35</td>
<td>-5.95</td>
<td>-0.21</td>
<td>3.12</td>
<td>2.90</td>
</tr>
<tr>
<td><strong>Month:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>January</td>
<td>-7.13</td>
<td>9.46</td>
<td>-1.76</td>
<td>-4.26</td>
<td>-3.39</td>
</tr>
<tr>
<td>February</td>
<td>-3.62</td>
<td>3.14</td>
<td>-0.02</td>
<td>-1.36</td>
<td>-1.74</td>
</tr>
<tr>
<td>March</td>
<td>-0.33</td>
<td>-0.16</td>
<td>0.30</td>
<td>0.20</td>
<td>-0.33</td>
</tr>
<tr>
<td>April</td>
<td>2.80</td>
<td>-3.21</td>
<td>0.04</td>
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<td>1.68</td>
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<td>-0.11</td>
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<td>1.23</td>
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<td>0.80</td>
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<td>-0.10</td>
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<tr>
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<td>-0.97</td>
<td>-1.70</td>
<td>-1.82</td>
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<td><strong>Weekday class:</strong></td>
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<td></td>
</tr>
<tr>
<td>Mon-Fri</td>
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<td>2.85</td>
<td>1.71</td>
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<td>-1.71</td>
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<td>-1.30</td>
<td>-0.51</td>
<td>1.56</td>
<td>0.26</td>
</tr>
</tbody>
</table>

VTI RAPPORT 363A
8. CONCLUDING REMARKS

In combination with the results of previous studies [1] and [2], representative results for passenger car speeds on German motorways are now available for 1982, and again for the years 1985 to 1987. Based on the existing BAB network in each case, the developments of the characteristics \( \bar{v}, v_{85} \) and \( P(v>130) \) are represented in Table 4.

Table 4: Characteristics of passenger car speed distributions in 1982, 1985 and 1987 weighted by the annual mileage on the entire BAB network

<table>
<thead>
<tr>
<th>Characteristics of passenger car speed distributions</th>
<th>Reference year</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \bar{v} ) [km/h]</td>
<td>112.3</td>
</tr>
<tr>
<td>( v_{85} ) [km/h]</td>
<td>139.2</td>
</tr>
<tr>
<td>( P(v&gt;130) ) [%]</td>
<td>25.0</td>
</tr>
</tbody>
</table>

Whereas the mean passenger car speed, \( \bar{v} \), shows a steady upward trend over the three reference years, the values of the characteristics \( v_{85} \) and \( P(v>130) \), determined within the framework of the large-scale exhaust gas test, have slumped to a certain extent. An explanation may be the urgent appeals to drivers at that time to reduce their speed for environmental reasons.

It should be noted that the values shown in Table 4 refer to a whole year in each case and may be subject to considerable fluctuations (cf. Chapter 7).

ACKNOWLEDGEMENT:

The author is indebted to Dipl.-Ing. Rüdiger Hotop from the German Federal Highway Research Institute who made available the speed-flow relationships and is co-author of references [1] and [10].

\( \text{VTI RAPPORT 363A} \)

\( 7^) \) The exact definition is found in Chapter 6.
References:

[1] D. Heidemann, R. Hotop:


[3] Bundesminister für Verkehr, Bonn (Edit.);
Deutsches Institut für Wirtschaftsforschung (DIW), Berlin (responsible for the contents):

[4] Bundesanstalt für Straßenwesen (Edit.):

[5] R. Hotop:

[6] Bundesanstalt für Straßenwesen (Edit.):


[8] U. Brannolte:

[9] D. Heidemann:

[10] D. Heidemann, R. Hotop:

Drivers' Attitudes and Beliefs Towards Speed Limits and Speeding on Dutch Motorways

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DRIVERS' ATTITUDES AND BELIEFS TOWARDS SPEED LIMITS AND SPEEDING ON DUTCH MOTORWAYS

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Abstract

In 1988, 1st of May, the legal speed limit for cars on most of the motorways in the Netherlands was raised from 100 to 120 km/h. On a small portion of the motorway network (about 17 per cent), especially on busy sections, the speed limit remained 100 km/h. The primary objective of this change of the legal speed limit, which has been accompanied with an increase of law enforcement efforts and several public information campaigns, actually was and still is to reduce the driving speed of motorists.

In order to evaluate the effects of these countermeasures on the opinions and motives of motorists towards speeding, three questionnaire surveys were carried out; the first was held in April 1988, the second in September 1988, and the third in April 1989. The questionnaires were mailed to independent samples of car drivers. The samples were taken by means of registratering number plates at several locations on motorways. At the same time, the actual driving speed was measured. The questionnaires consisted of items concerning reported behaviour, attitudes and beliefs towards speeding, the legal speed limits and law enforcement.

On the basis of the data with regard to both the main reason for driving a car on working days and the owner of the car, four groups of car users could be distinguished, which systematically differed on most of the relevant variables. The groups are:
- private drivers, i.e. car drivers who mainly use their car for private purposes like shopping and visiting;
- commuters by car;
- business drivers like salesmen with a private car;
- business drivers with a company car.

The results of the surveys show that the private drivers speed the least (recorded as well as reported), they have the most favourable attitudes towards the speed limits and police enforcement, and they have the most unfavourable attitudes towards speeding. On the other hand, the business drivers with a company car speed the most (recorded as well as reported), they have the most unfavourable attitudes towards the speed limits and police enforcement and they have the most favourable attitudes towards speeding. The other two groups take a position between these two extremes.
In comparison with the pretest in April 1988 the registered mean driving speed of the respondents was much lower in September 1988. Especially the percentage of motorists driving at high speeds was reduced. In April 1989 however, the mean driving speed turned out to be back at the pretest level, although the range of measured speed was smaller.

With regard to the attitudes and motives towards speeding, both positive and negative results were found. A positive result was that police enforcement formed a stronger motive not to speed after the conversion of the speed limits. This effect was found with all four groups of car users. As a contrast, especially the business drivers perceived speeding as less risky in April 1989. Also the pleasure in driving was a stronger motive to speed for them.
1. INTRODUCTION

In 1973, at the time of the international oil crisis, a legal speed limit of 100 km/h was implemented for cars on the Dutch motorways. At the outset this limit was obeyed quite well. During the following decade however, an almost yearly increase of the mean speed of the motorists could be observed. In the mid-eighties a vast majority of motorists exceeded the limit. As the speed limit was hardly accepted at that time and taking into account the available manpower of the police force, enforcing the 100 km/h limit was supposed to be impossible.

On May 1st, 1988, the speed limit was raised to 120 km/h on most of the motorways. Especially on busy sections of the motorway network, the limit remained 100 km/h. As the 120 km/h limit was found to be far more acceptable (Vogel & Rothengatter, 1985), the introduction of this new limit aimed to control motorists' driving speed. The objective of this change, which has been accompanied by a sharp increase in law enforcement efforts and several public information campaigns, was and still is to reduce the mean driving speed of motorists. Excessively high speeds in particular should be reduced, owing to the detrimental effects on traffic safety and air pollution.

During the first year following the implementation of the new limit, several studies were carried out, with support of the Ministry of Transport, to evaluate the effects of the countermeasures with respect to driving speed and traffic safety. As part of this evaluation research, the Traffic Research Centre conducted survey studies amongst motorists in order to determine their opinions towards the speed limit and towards speeding both before, during, and after the first year with the 120 km/h limit (Rooijers, 1989a). In addition the surveys aimed to distinguish different groups of car drivers with regard to driving speed and their attitudes and motives towards speeding.

2. METHOD

Three questionnaire surveys were carried out. The first survey was conducted in April 1988, preceding the implementation of the new limit. The second survey was conducted in September 1988, and the third in April 1989, about a year after the introduction of the 120 km/h limit. For each survey a large number of private car licence plates were registered at several locations on the motorways. At the same time, the actual driving speed was measured by radar registration. From this pool of car drivers a speed-dependent stratified sample was taken, to whom the questionnaire was mailed. After about one week all addresses received a reminder.

The questionnaires were mostly the same for all three surveys. Apart from some general demographic items, each questionnaire consisted of the following parts:

**Behaviour**
The respondents were requested to report their habitual speed on motorways as well as their preferred speed.

**Opinions about the speed limits and about police enforcement**
Both the 120 km/h limit and the unchanged 100 km/h speed limit on a number of motorway sections were evaluated. In addition the attitudes towards police enforcement of driving speed on motorways were measured.
Attitude towards speeding
The respondents were asked to evaluate the target behaviour "exceeding 120 km/h on the motorway" on three bipolar rating scales. The sum of the three ratings constituted the individual attitude score.

Behavioural motives to speed
In the questionnaire 19 potential consequences of speeding, both positive and negative, were listed. Examples of the consequences listed are 'time saving', 'more pleasure in driving', 'higher risks', 'higher chance to be caught by the police', and 'more air pollution'. For each of these consequences two ratings were obtained. Firstly, the respondents were asked to evaluate each consequence. Secondly, they were requested to estimate the likelihood of each consequence in case of speeding. Both ratings were made on bipolar rating scales. The multiplications of these two ratings for each consequence were considered as the individual behavioural motives to speed (positive scores) or not to speed (negative scores).

Normative motives to speed
As social norms can have a significant impact on behaviour as well, we also studied the relevance of normative beliefs with respect to speeding. To do this, seven possible reference groups were listed in the questionnaire (e.g. relatives and friends, colleagues, the police). Again, for each reference group two ratings were obtained. Firstly, the respondents were requested to estimate whether exceeding the limit would be approved or disapproved by the reference group. Secondly, they were asked to indicate the extent to which they are motivated to comply with the opinions of those in the reference group. In the same manner as with the behavioural motives, the multiplications of both ratings were considered as the individual normative motives to speed or not to speed.

Opinions about public information campaigns
In the second and third survey a few additional items were included concerning the publicity campaigns.

The behavioural analysis technique as described above with respect to attitudes and behavioural and normative motives to speed is based on Fishbein and Ajzen's theory of reasoned action (1975; Ajzen & Fishbein 1980).

3. RESULTS
The response to the three surveys was about 60 per cent. In relation to the number of questionnaires mailed per speed category, especially people in the extremely low speed classes and the extremely high speed classes were somewhat underrepresented.

3.1 Recorded and reported speed
The first objective of the study was to specify the group of car drivers with the highest driving speed. Based on the data with regard to the main reason for driving a car on working days and the owner of the car, four groups of car users could be distinguished, which significantly differed on the recorded driving speed. These groups are:
- private drivers, i.e. car drivers who mainly use their car for private purposes such as shopping and visiting;
- commuters by car;
- business drivers, such as salesmen, using their own car;
- business drivers using a company car.

The private drivers rarely exceed the speed limit. In contrast, the business drivers using a company car exceed the speed limit regularly or even most of the time. The other two groups take a position between these two extremes.
Figure 1 shows the mean recorded driving speed for each group as well as the mean reported speed and the mean preferred speed. These data refer to the first survey only. As can be seen in the figure, the groups also differ systematically on the reported speed and the preferred speed. In addition, the reported driving speed turns out to be higher than the registered driving speed. Moreover, all four groups prefer to travel at higher speeds than those at which they normally drive.

![Graph of recorded, reported, and preferred driving speed](image)

Figure 1: Recorded, reported and preferred driving speed on motorways prior to the introduction of the 120 km/h limit

As both the recorded speed and the reported speed were known for each respondent, the correlation between these two variables could be calculated. Only a moderate positive correlation (Pearson’s product-moment correlation) was found ($r = .48, p < .001$). This means that less than 25 per cent of the variance of the one speed measure can be explained by the other. A much higher correlation was found between the reported habitual driving speed and the speed at which the respondents prefer to drive ($r = .86, p < .001$).

Figure 2 shows the mean registered driving speed distinguished by the group of car users and by the survey. With the second survey, conducted five months after the implementation of the 120 km/h limit, the mean driving speed of business drivers was considerably lower than before the introduction of the new limit. The mean speed of the commuters remained unchanged, while the private motorists drove slightly faster. At the time of the third survey all four groups of car users show an increase in the mean driving speed. The largest increments are found, however, among the business drivers.
Statistical analyses of the speed data showed an overall main effect for moment of registration ($F(2,2240) = 7.8, p < .001$) and for car user group ($F(3,2240) = 48.8, p < .001$). At the time of the second survey the overall mean driving speed was significantly, although marginally, reduced (110 vs 112 km/h, $t = -3.2, p < .01$). No difference could be observed between the first and the third registration. As can be concluded from figure 2, however, the driving speed of business drivers turned out to be still lower a year after the introduction of the 120 km/h limit. Moreover, as both the private motorists and the commuters by car drove at higher speeds than before, the speed range was considerably smaller, which is supposed to have a positive effect on road safety. In accordance with these different trends among the four groups of car users, a significant interaction effect was found between moment of registration and car user group ($F(6,2240) = 8.2, p < .001$).

With regard to the reported driving speed and the preferred speed no effect for the moment of registration could be observed. In contrast, the systematic differences among the four car user groups on these variables were ever greater ($F(3,2240) = 111.7, p < .001$) and $F(3,2240) = 112.4, p < .001$ respectively).

3.2 Opinions about speed limits and police enforcement

Figure 3 shows the mean rating with respect to the evaluation of the 120 km/h limit distinguished by car user group and by measurement. Again a significant main effect for measurement is found ($F(2,2143) = 44.4, p < .001$). Before the introduction, the 120 km/h limit was rated more positively by the motorists. Probably of more relevance is the observance of a significant interaction effect between survey and car user group ($F(6,2143) = 18.3, p < .001$). Prior to the introduction of the limit, the attitude towards the limit was most positive among the business drivers using a company car and least positive among the private drivers. At the time of the third survey however, the rankorder was totally reversed. The private drivers rated the limit most positively, followed in descending order by the commuters by car, the business driver using their private car, and, at last, the business drivers using a company car.
Figure 3: Attitudes towards 120 km/h speed limit

With respect to the 100 km/h limit on certain motorway sections, a significant main effect for the survey ($F(2,2143) = 46.9, p < .001$) as well as an interaction effect between survey and car user group ($F(6,2320) = 3.1, p < .01$) could be observed. This locally unchanged speed limit was also valued more positively at the time of the first survey. In particular the business drivers using a company car rated the limit more negatively at the time of the latter surveys.

Significant and systematic differences among the four groups of car users are also found with regard to police enforcement, in particular with regard to the attitude towards police enforcement of speeding behaviour ($F(3,1236) = 12.7, p < .001$), desirable changes in the amount of enforcement efforts ($F(3,1240) = 17.9, p < .001$), and the motivation to drive faster when the chance of being caught would be zero ($F(3,1243) = 24.2, p < .001$). Police enforcement of speeding behaviour is rated most positively by the private drivers and least positively by the business drivers using a company car. In addition most private drivers express a desire for the amount of enforcement efforts to be extended whereas most business drivers using a company car have no such desire to extend these efforts. Moreover, the latter group is more highly motivated to drive faster when the chance of being caught would be zero.

The latter two variables also show a significant main effect for survey ($F(2,2309) = 12.7, p < .001$) and $F(2,2309) = 29.7, p < .001$). Probably as a result of the increased enforcement efforts since the introduction of the 120 km/h limit, less respondents reported a preference for more enforcement activities at the time of the second and the third survey. In addition the motivation to drive faster when the chance of being caught would be zero was stronger after the implementation of the new limit.

3.3 Attitudes towards speeding

Table 1 shows the mean rating with respect to the attitude towards exceeding the 120 km/h limit on motorways, distinguished by car user group and by survey. Again, a significant main effect for group is found ($F(3,1439) = 50.6, p < .001$). As the data in table 1 reveal, the private drivers have the most negative attitude towards speeding, whereas the business drivers using...
a company car have the most positive attitude. The other two groups take a position between these two extremes.

Table 1. Attitudes towards exceeding 120 km/h on motorways

<table>
<thead>
<tr>
<th>groups:</th>
<th>1. private drivers</th>
<th>2. commuters by car</th>
<th>3. business drivers using their private car</th>
<th>4. business drivers using a company car</th>
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</thead>
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<td>survey</td>
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<td>4</td>
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<tr>
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<td>.45</td>
<td>1.33</td>
<td>2.79</td>
</tr>
<tr>
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<td>.20</td>
<td>1.13</td>
<td>2.28</td>
</tr>
<tr>
<td>III</td>
<td>-2.67</td>
<td>.18</td>
<td>.79</td>
<td>2.50</td>
</tr>
</tbody>
</table>

Although the mean attitude rating shows a downward tendency within each group, no significant main effect for survey, nor significant interaction effects could be observed.

3.4 Behavioural motives to speed or not to speed

Within each survey the respondents were asked to evaluate nineteen potential consequences with regard to speeding. In addition they were requested to estimate the likelihood of the consequences in the case of exceeding 120 km/h on motorways. The multiplication of these two ratings for each consequence is considered as the individual motivation to speed in relation to that particular consequence.

On sixteen of the behavioural motive variables significant and systematic differences are found among the car user groups. In general it turns out to be that the positive consequences of speeding (e.g. time savings, pleasure in driving) are rated most positively by business drivers using a company car and least positively by private drivers. On the other hand the negative consequences of speeding (e.g. higher accident risk, chance of being caught by the police, more air pollution) are rated most negatively by private drivers and least negatively by business drivers with a company car.

Apart from the group effects, a main effect for measurement was observed on fourteen variables. The direction of these effects however was less clear cut. That is to say, from the standpoint of road safety, both positive and negative effects were found. In an attempt to make the mixed results more comprehensible, factor analysis was applied on the individual motive ratings (Principal Components Analysis with VARIMAX rotation). The best solution seemed to be the extraction of three factors, although the explained variance was relatively low (42 per cent). These factors were labeled 'Risks', 'Pleasure', and 'Enforcement'. Subsequently, three new variables were formed by summing up all ratings, weighed by their respective factor loadings. Table 2 shows the mean scores on these newly composed variables and the results of the analysis of variance.
Table 2. Mean ratings on newly composed speed motivation variables

<table>
<thead>
<tr>
<th>Groups</th>
<th>Effects</th>
</tr>
</thead>
<tbody>
<tr>
<td>0. all respondents</td>
<td>F1 = 7.7, p &lt; .001</td>
</tr>
<tr>
<td>1. private drivers</td>
<td></td>
</tr>
<tr>
<td>2. commuters by car</td>
<td>F2 = 37.2, p &lt; .001</td>
</tr>
<tr>
<td>3. business drivers using their private car</td>
<td>F3 = 2.9, p &lt; .01</td>
</tr>
<tr>
<td>4. business drivers using a company car</td>
<td></td>
</tr>
</tbody>
</table>

As the data in Table 2 show, on all three newly composed variables significant main effects for the survey are found. The effect on the risks factor can be seen as being negative. Following the introduction of the 120 km/h limit, risk-related consequences are rated less negatively by the respondents, except for the private drivers, but especially so by the business drivers.

In contrast, the effect on the enforcement factor can be considered as being positive. Enforcement-related consequences turn out to be rated more negatively by all groups of car users, but especially by the private drivers.

With regard to the pleasure factor, initially a slightly positive trend can be observed. At the time of the second survey pleasure-related consequences seem to be judged less positively. With the third survey, however, a reversed effect is found. Business drivers using a company car in particular, seem to have more fun in speeding.

3.5 Normative motives to speed or not to speed

Seven possible reference groups were listed in the questionnaire (relatives/friends, passengers, colleagues/boss, traffic experts, other motorists, the police, and the authorities). Recall that the respondents were requested to estimate the opinions of these groups about exceeding 120 km/h on motorways and to report their motivation to comply with the opinions of those in the reference group. The multiplication of these two ratings for each reference group is considered as the individual's normative motive to speed with regard to that particular group.

On all seven normative motive variables again significant and systematic differences are found among the car user groups. In general, according to the private drivers, all listed reference groups disapprove of speeding. In contrast, according to the business drivers, exceeding the limit is approved of by most of the reference groups. Only the police and the auth-
orities would disapprove of it. In addition, the private drivers are more inclined to take the opinions of these other people into account than the other car user groups.

A significant main effect for measurement is found on four variables, namely colleagues, other motorists, the police, and the authorities. All these effects are in the desired direction. That is to say, the normative motive ratings are less positive c.q. more negative after the introduction of the 120 km/h limit, especially at the time of the third survey. In other words, according to the respondents, exceeding the limit was more disapproved of by the mentioned reference groups after the introduction than before. The largest differences in rating were found with respect to the police and to the authorities. Particularly with respect to the police, the respondents were also more motivated to comply.

The separate normative motive ratings are added up for each respondent to determine his or her 'overall' normative motive to speed. The mean scores are shown in table 3.

<table>
<thead>
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<th>3</th>
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<td>-11.0</td>
<td>-1.1</td>
</tr>
<tr>
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<td>-14.2</td>
<td>-28.3</td>
<td>-14.7</td>
<td>-10.0</td>
<td>-4.6</td>
</tr>
<tr>
<td>III</td>
<td>-18.2</td>
<td>-29.3</td>
<td>-19.2</td>
<td>-15.1</td>
<td>-13.1</td>
</tr>
</tbody>
</table>

4. DISCUSSION

First of all, the results described in the preceding sections clearly indicate that the different groups of car users form a relevant distinction among the population of car drivers. Not only do these groups differ systematically on behavioural measures, they also turn out to differ on almost all other variables relating to driving speed on motorways. Private drivers, i.e. people who mainly use their car on working days for private purposes, drive at the slowest speed (recorded as well as reported) and rate the speed limits and the enforcement efforts most positively. In addition, possible positive consequences of speeding are rated least positively, possible negative consequences are rated most negatively and most reference groups are perceived as disapproving of speeding. In contrast, business drivers using a company car speed the most and rate the limits and police enforcement least positively. In addition, the positive consequences of speeding are rated most positively, the negative consequences are rated most negatively and exceeding the limit is scarcely disapproved of by their reference groups. The other two groups, commuters by car and business drivers using their private car, take a position between these two extremes on most of the variables.

The position of private drivers on the variables might be not surprising. The age level of these motorists is generally higher, a substantial number of them are retired, and older people tend to drive slower and to behave more normatively (Brouwer, 1989). Perhaps more surprising are the systematic differences among the other three groups, in particular the differences between the two groups of business drivers. Obviously, the mere fact that the car the one drives is not the private car and that the costs of using the car have do not to be paid for, leads not only to higher driving speeds, but to undesirable attitudes and beliefs towards speeding as well. Unfortunately, employees of more and more trade concerns seem to have a company or leased car at their disposal.
The introduction of the 120 km/h limit on the Dutch motorways in May, 1988, combined with a sharp increase in the enforcement efforts and several public information campaigns, was aimed at reducing the driving speed of motorists. High speeds in particular, should be reduced. As the recorded speed data show, the mean driving speed of the business drivers was indeed considerably lower at the time of the second survey. At the time of the third survey their mean driving speed was almost back to pretest level. Hence, the implementation of the 120 km/h limit and the additional measures of enforcement and publicity had a positive though temporary effect. Apart from the speed reduction amongst the business drivers, a reversed effect was found amongst the private drivers. They tended to drive faster. This negative 'side-effect' could be expected, however, seeing as they were allowed to drive faster. One year after the introduction of the 120 km/h limit, the overall mean driving speed turned out therefore to be at the same level as before the introduction. However, in view of the observed opposite effects, the speed range was smaller, which can be considered as a positive result.

A remarkable effect was found with respect to the attitudes towards the 120 km/h limit. Before the introduction, this limit was rated most positively by the business drivers using a company car and least positively by the private drivers. In contrast, one year later, the limit was rated most positively by the private drivers and least positively by the business drivers. The reason for this turnover effect is not totally clear. Perhaps the business drivers felt more restrained as a result of the enforcement efforts than they had expected to feel beforehand. For them, the increment of the speed limit did not mean a legalization of their habitual driving speed, as they had expected, but, in contrast, it meant more restraints upon their driving behaviour.

The attitudes towards speeding are not affected by the measures, although there seems to be a slight tendency within the four car user groups towards less positive c.q. more negative thoughts of exceeding the limit.

With respect to the behavioural and normative motives to speed, both positive and negative effects for the moment of survey are found. The positive effect is that, in general, enforcement-related consequences of speeding form stronger motives for not speeding after the introduction of the 120 km/h limit. In addition, the perceived social norms towards exceeding the limit turn out to be more negative. These effects can be observed within all groups of car users, but the effects are strongest among the private drivers. However, with regard to the risk-related consequences of speeding a negative effect is found. These consequences seem to form weaker motives for not speeding after the introduction of the new limit, especially among the business drivers. Moreover, the business drivers seem to enjoy speeding after the introduction more than before. Perhaps these negative effects are caused by reaction of the motorists feeling constrained in their speed choice.

These results indicate that the desirable effects are especially found among those motorists already behaving most normatively. The car drivers with the highest habitual driving speed and with the most positive attitudes and beliefs towards speeding turn out to be the least affected in the desirable direction. They did show a sizable reduction in their actual driving speed, but only temporarily. Negative consequences of speeding, such as accident risk and air pollution, are for them not very persuasive arguments. Only the chance of being caught seems to be an effective factor. Unfortunately, however, this chance is far too small to affect the business drivers' behaviour continuously. In advance of the implementation of the 120 km/h limit, the limit change as well as the increase in the enforcement efforts received a lot of (free) publicity in the media. As a result of this extensive attention, the subjective chance of being caught was probably considerably increased. For this reason the (business) car drivers might have slowed down during the first few months after the implementation. However, as the drivers may have noticed that the actual risk of being caught was much lower than they had expected it to be beforehand, they gradually raised their driving speed. Subsequent publicity campaigns have not helped. As a consequence of low intensity of the public information activities, the target group (the business drivers) was hardly reached (Rooijers, 1989b).
The question still remains what must be done to convince the business drivers to slow down. Some suggest reintroducing the old 100 km/h limit or introducing an even lower limit. But how would that limit be enforced? A more plausible solution would be to enforce the existing speed limit in a more efficient and effective way. Drivers should be confronted with obtrusive speed control activities more often. Apart from this, intensive public information campaigns should be carried out on a regular basis and especially directed towards business drivers. In addition, the employers, who form an important intermediary group, should be persuaded to co-operate in efforts to reduce the driving speed of their employees.

REFERENCES


