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Sustainability Effects of Traffic Management Systems

Sven Maerivoet† and Bart De Moor†

March 2006

Final report for the DWTC PODO-II CP/40 project

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†Katholieke Universiteit Leuven
Department of Electrical Engineering ESAT-SCD (SISTA)
Kasteelpark Arenberg 10, 3001 Leuven, Belgium
Phone: (+32) (0) 16 32 86 64 Fax: (+32) (0) 16 32 19 70
E-mail: {*sven.maerivoet,bart.demoor*}*@esat.kuleuven.be*
WWW: *http://www.esat.kuleuven.be/scd*
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FINAL REPORT

SUSTAINABILITY EFFECTS OF TRAFFIC MANAGEMENT SYSTEMS

CP/40

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Rue de la Science 8
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B-1000 Brussels
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Tel: +32 (0)2 238 34 11 – Fax: +32 (0)2 230 59 12
http://www.belspo.be

Contact person:
Mrs Hilde Van Dongen
Secretariat: +32 (0)2 238 37 61

SCIENTIFIC SUPPORT PLAN FOR A SUSTAINABLE DEVELOPMENT POLICY
(SPSD II)

Part 1:
Sustainable production and consumption patterns

FINAL REPORT

SUSTAINABILITY EFFECTS OF TRAFFIC MANAGEMENT SYSTEMS

CP/40

Names of the promoters - Institution/Organisation
prof. dr. ir. Bart De Moor – Katholieke Universiteit Leuven
prof. ir. Ben Immers – Katholieke Universiteit Leuven
prof. dr. Stef Proost – Katholieke Universiteit Leuven
prof. dr. ir. René Boel – Universiteit Gent
prof. dr. ir. Georges Bastin – Université Catholique de Louvain

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ESAT SISTA

ENERGY TRANSPORT & ENVIRONMENT

UNIVERSITEIT GENT SYSTeMS

CESAME Centre for Systems Engineering and Applied Mechanics

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Introduction

This report describes the methodology and results related to a study for using advanced traffic management systems (ATMS) in order to expand the capacity of the Belgian road network. ATMS consist of actuators that interact with the traffic stream or the road and of control strategies. These control strategies are specifically developed in order to maximize the sustainability. By using the correct optimisation techniques and simulation models, which find the balance between accuracy and computation time, the result, which will take the form of a control strategy, will be seriously improved. At a national scope, the developed traffic models can be implemented for the Belgian traffic situation, which will give rise to more optimal flowing traffic, with an eye for certain characteristics of sustainability.

Context and summary

When thinking in a sustainable mobility framework, one approach could be to limit the traffic demand and to balance this demand over different traffic modes. As a complementary approach, one could also try to optimise the use of the existing infrastructure. In this project we will look into detail at the second approach. More specifically, we will look at advanced traffic management systems (ATMS) as a means to enlarge the traffic capacity of the Belgian highway network without constructing new roads.

An inventarisation of the available and relevant data will be made. For traffic measurement data we look at the currently implemented technologies as there are loop detectors, traffic cameras, . . . These technologies all provide data with different levels of accuracy. In order to monitor traffic (traffic situation, incident detection, . . .) or to model traffic, attention has to be paid to data consistency. Conflicting or missing measurements have to be corrected or estimated. For accurate modelling additional data is needed. The weather has definitely a non-negligible effect on traffic but also maintenance works on the highways, incidents influence, . . . need to be taken into account.

As already mentioned, traffic measurements are important for building simulation models of highway networks. The measurement data are used to estimate the model parameters. During traffic simulation the inputs of the model are typically provided to the model as Origin-Destination (OD) matrices. The dynamic estimation of OD matrices based upon the available traffic measurements is currently an area of research. After the model and the OD matrices are determined, a thorough model validation will be executed.

In order to be able to assess the 'quality' of a simulated traffic situation, we need to define goals we would like to achieve or stated in control terms: a cost function. In the scope of this project on sustainable mobility, a definition of the cost function could include penalisations for emission (environmental cost), congestion (socio-economic cost), noise, dangerous situations (like shock waves), . . . The cost function is expressed in terms of the states of the model and can be evaluated during simulation. The exact definition of the cost function to be used will be subject to research and alternatives will be evaluated.

An ATMS consists of actuators that interact with the highway traffic flows and of a control strategy that attempts to minimise the cost function. ATMS implementing ramp metering, velocity harmonisation, . . . are to be studied in detail. Starting with controllers for one actuator, the research will evolve towards co-ordinated control of multiple actuators. Using the right optimisation techniques as well as simulation models that find the correct trade-off between level of detail and computational complexity will greatly influence the optimality of the resulting controller.

Overview of this report

The research of our project is outlined as follows: in the first part, we discuss the scientific methodology which is based around 7 clearly defined aspects:

- The inventarisation of technology and monitoring systems
- Modelling
- Dynamic OD-estimation
- Model fitting and validation
- Sustainable cost function
- Control techniques
- Optimisation

Each of these aspects is subsequently dealed with in detail in the second part of this report. As each aspect builds upon the knowledgde obtained in the previous one, the final aspect *"Optimisation"* will integrate these. It shows the possibilities that can be achieved by controlling a traffic system with respect to a socio-economical and environmental trade-off.

The third part of the project briefly discusses future prospects and future planning, after which the final part follows. This latter part contains the annexes that constitute the references, as well a list of papers published during the course of the project's duration.

Project promotors and affiliations

1. SISTA – **Signals, Identification, System Theory and Automation**, Katholieke Universiteit Leuven
   prof. dr. ir. Bart De Moor
2. TPHE – **Transportation Planning and Highway Engineering**, Katholieke Universiteit Leuven
   prof. ir. Ben Immers
3. ETE – **Energy, Transport and Environment**, Katholieke Universiteit Leuven
   prof. dr. Stef Proost
4. SYSTeMS – **Department of Electrical Energy, Systems and Automation**, Universiteit Gent
   prof. dr. ir. René Boel

5. CESAME – **Center for Systems Engineering and Applied Mathematics**, Université Catholique de Louvain
   prof. dr. ir. Guy Campion
   prof. dr. ir. Georges Bastin

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Part I

Description of the scientific methodology

In this project, we studied the traffic on the Belgian highway network. The goal of this study was the design of sustainable control strategies for Advanced Traffic Management Systems (ATMS) on highways. This involves a lot of research in different fields. ATMS are able to deal with the dynamics of the traffic situation during congestion, incidents, ... and attempt to steer the highway system towards a more optimal state. The definition of an optimal state is expressed in terms of a cost function that is minimised. In the framework of sustainable mobility, the cost function contains terms that penalise environmental, ecological, economical and social costs. In order to assess the value of the cost function for various traffic situations resulting from different control strategies and different scenarios we will develop dynamical traffic models. These dynamical traffic models are calibrated and simulated using highway traffic measurements which we have available. The simulation of the traffic models enables us to estimate the current state and to predict the future state of the highway network. By simulating the dynamical traffic models and by using optimisation techniques we can determine optimal strategies to control the ATMS.

The distinguished project steps are as follows:

(1) The inventarisation of technology and monitoring systems

In this first step the available operational and planned advanced traffic management systems (ATMS) in Belgium will be listed and studied. Also a study of ATMS in development abroad will be made. We mention ramp metering, velocity harmonisation and route directives as the current state-of-the-art ATMS. The ATMS that will be taken into account in step 6 of this project will be chosen based on our inventory. Secondly the traffic monitoring systems and the available and relevant traffic data will be inventoried. Currently the traffic on highways in Belgium is monitored mainly using loop detectors and cameras. These provide measurements of the number of vehicles passing, the highway occupancy and the average velocity of the vehicles for every minute of the day. Through our contacts with the instances in charge of these measurements we can obtain these data for use within the project. Other potentially interesting technologies like probe vehicles, triangulation of locations of mobile phones by mobile operators, ... will be included in the survey as well. Contacts with companies in this field like Webraska, mobile operators, ... are already established within the consortium.All these technologies provide data with different levels of accuracy. In order to monitor traffic (traffic situation, incident detection, ...) or to model traffic, attention has to be paid to data consistency. Conflicting measurements need to be 'fused' together while missing measurements need to be estimated. The quality of the measurements is determined by their variance. A thorough statistical study of the measurements is imperative. It is clear that the weather has an influence on traffic but also maintenance works on the highways, incidents, ... need to be inventoried and be taken into account. This research leads towards the development of data processing techniques on a Belgian level.

(2) Modelling

A second step in this project is to set up a modelling framework. Different traffic models exist, and the choice of a model depends on the application it is used for. The members of the consortium have built up expertise in both macroscopic and microscopic traffic simulation models within the framework of the DWTC project 'The congestion problem in Belgium: mathematical models, analysis, simulation, control and actions' (MD/01/024-025) where the E17 Ghent-Antwerp was modelled. In this project we will take modelling to another level: the experience of the previous project will be used to enlarge the geographical scope and to model the Belgian highway network using dynamical models. The computational complexity of the models will be investigated in order to meet the time requirements imposed by the grown network. The integration of different types of models (e.g. micro- and macro models) will be investigated in order to combine the advantages of both techniques e.g. trade-off between computational complexity and level of detail.The results of this modelling research can be exploited in various ways. Dynamical models can be used for model based fault detection and isolation as described recently in the context of timed discrete event systems. This technique can be used for incident detection on a freeway network. Based on the data and the dynamical model, estimations of the current and predictions of the future traffic state can be made. This allows for simulation of different scenarios and to act accordingly. Model based simulations of the cost of different traffic regimes can be performed and the impact of ATMS on traffic states can be assessed. This leads to optimisation of control techniques as described in detail in part 7.

(3) Dynamic OD-estimation

A dynamic traffic flow model calculates the dynamic traffic pattern based on the input conditions. The traffic demand, infrastructure, weather conditions, advanced traffic management systems and the control strategies of these systems are important input variables. Traffic demand is specified as dynamic origin-destination (OD) matrices which depend on the time of day, the weather conditions, big events, . . . These matrices contain the amount of vehicles that want to make a trip from an origin to a destination under the current traffic state. The estimation of these dynamic OD tables is currently an interesting area of research. Static OD estimation techniques based on socio-economic data will be combined with measured flow data. The availability of new data like trajectories of probe vehicles will allow for an increased accuracy of the OD matrix estimations. Knowledge of OD-tables in several traffic regimes (normal, congestion, incidents, . . .) and for several time periods (now, within a few years, . . .) is necessary for accurate modelling. When developing a real-time traffic model, the processing of the traffic data and the estimation of the OD-tables which serve as input to the model need to happen very fast and automatically.

(4) Model fitting and validation

The parameters of the dynamic highway traffic models are tuned using real-life datasets during the model-fitting phase. The set of parameters that causes the dynamical model to mimic the real traffic conditions best is looked for. Within this step several historical traffic patterns (e.g.: incidents, congestion, holiday, . . .) will be used to test and to improve the model. After calibration of the model, the model needs to be thoroughly validated using new datasets. A good calibration of the highway network model requires:

- The calibration and validation of the estimated dynamic OD matrices as discussed in step 3.
- The calibration and the validation of the dynamic traffic model itself.

The ATMS will be used as actuators to interact with the traffic. These ATMS need to be calibrated and validated in detail as well.

(5) Sustainable cost function

When trying to reach a more sustainable traffic state, we need to evaluate a dynamic traffic pattern. In this project we translate the goals we would like to achieve into detailed quantities which are function of the traffic/model state. In economic terms a cost function is defined. The occurrence of congestion, emission, noise, unsafety, unreliability, unaccessibility, . . . is rendered as a cost function and every contribution to this cost can be weighed separately, depending on the exact definition of sustainability. The contributions to the cost function are expressed in terms of the traffic states. The modelled traffic patterns can be evaluated, and in a next step be optimised, with the help of the cost function. The exact definition of the cost function to be used to induce sustainable traffic operation in Belgium will be subject to research and alternatives will be evaluated. The new approach lies in the calculation of a cost function based on dynamic traffic states instead of using the traditional static model.

(6) Control techniques

An ATMS consists of actuators that interact with the highway traffic flows and of a control strategy that responds to the state of the traffic. Out of the inventory of ATMS from step 1, a set of systems will be chosen. Possible ATMS are: ramp metering, velocity harmonisation, dynamic route information, . . . After calibration of the ATMS we can simulate their behaviour. In practise the implementation of ATMS creates side effects e.g. a ramp metered on-ramp will create a local re-routing of traffic. In order to anticipate these side effects it is necessary to look into co-ordinated control of multiple ATMS. We will study how ATMS influence each other in order to develop co-ordinated control strategies on a Belgian scale.

(7) Optimisation

In this project, we look at optimal control of ramp metering, as well as other ATMS. The goal is to minimise the cost function in order to achieve what is defined as the optimal or maximally sustainable traffic state on the highway network. This can be realised using optimisation techniques that look for the traffic states which minimise the cost function by steering the ATMS actuators. We are dealing with non-linear constrained optimisation since the traffic model is not linear and the relation between the states described in the model can not be violated. Using the right optimisation techniques as well as simulation models that find the correct trade-off between level of detail and computational complexity will greatly influence the optimality of the resulting controller(s). It is clear that the definition of the cost function greatly influences the sustainability of the solution found.

Part II

Description of the intermediary results, preliminary conclusions and recommendations

Chapter 1

Inventarisation of technology and monitoring systems

In this chapter, we first discuss the ATMSs that are currently under consideration in Belgium, as well as abroad (in The Netherlands), followed by an overview of the available traffic data collection systems.

1.1 Available advanced traffic management systems

At this moment, only few ATMSs are employed in Belgium:

- There is one ramp metering installation at the E314 on-ramp in Kessel-Lo (Leuven); the device is only operational on weekdays between 06h00 and 10h00 if the occupancy is more than 18%, i.e., an average level of traffic [18].
- At another level, an overtaking prohibition for trucks was implemented by the government (from january 2004 on); it is only applied during rainy weather (but the concept of ’rain’ hasn’t been legally ’defined’ yet in this context).

We have been extensively investigating the use of ramp metering as a control methodology using model predictive control [66]; the results are presented in sections 2.2.2, and 6.4. The remainder of this section gives a more elaborate discussion on the effects of an overtaking prohibition for trucks, and the consequences of applying a dynamic road pricing scheme in order to prevent congestion due to blocking back effects.

(I) Overtaking prohibition for trucks

In highway traffic, the interactions between trucks and passenger cars are highly important. Therefore, we investigated whether an overtaking prohibition for trucks would be worthwhile [28, 55]. To that end, a great amount of situations was modelled, using our dynamic, heterogeneous traffic flow model (see section 2.2.1 for more details). The possible gain in travel time served as a criterion. Furthermore, a distinction was made between the appreciation of time between trucks and passenger cars.

The results of this study state that:

- an overtaking prohibition for trucks becomes more interesting if the traffic demand of passenger cars increases,

- the possible benefit decreases as soon as the traffic demand of passenger cars exceeds the traffic lane capacity and/or as soon as the traffic demand of trucks increases (with a constant traffic demand of passenger cars) and/or as soon as number of traffic lanes increases.

(II) Dynamic road pricing

For the justification of dynamic road-pricing, we combined two models. For the description of the traffic demand, we used the bottleneck model of Vickrey [69] and for the traffic propagation, the LWR model [42, 45]. With this combined model, it is possible to analytically calculate the user and system optimum of a small network [53]. Application of an optimal, dynamic toll system can make traffic propagation without congestion possible.

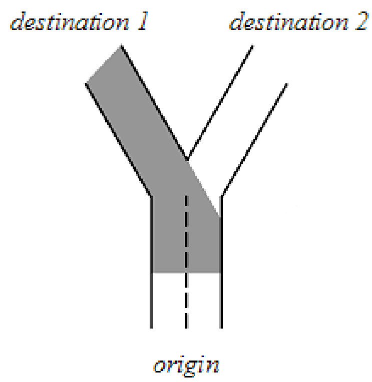


Figure 1.1: Travellers going from 'origin' to 'destination 2' may encounter congestion that originates from 'destination 1' which is not their bottleneck.

This study provided an unsuspected insight. The blocking-back of traffic jams makes it possible that travellers experience congestion, though they do not have to pass through the actual bottleneck (see Figure 1.1). The application of optimal road-pricing decreases the travel costs for those travellers. As a result of this, the traffic demand increases in the rush hour, without causing congestion. Consequently, we can conclude that road pricing can have a positive effect on the network performance.

Both the previously discussed ATMSs are based on using the first order macroscopic LWR model for heterogeneous vehicular traffic (refer to section 2.2.1 for more details) [54, 52].

Another more local system is intelligent speed adaption (ISA), which was part of a pilot project in the city of Gent. For more details, we refer to the work done at the Universiteit Gent in cooperation with the *"Centrum voor Duurzame Ontwikkeling"* [68].

Recently, a study was finished discussing another possible measure that is to be used in The Netherlands: model predictive control applied to dynamical speed limits [2]. The main results of this research implied the following:

- the use of dynamic speed limits is very suitable to prevent or eliminate shock waves on the highway,
- dynamic speed limits that are used to limit the speed as well as the flow can, in combination with other measures (such as ramp metering), result in a better prevention/resolvement of traffic jams,
- model predictive control proves to be very suitable in solving traffic control problems,

- and the improvement of the total time spent by all the vehicles in a traffic network that is achieved by traffic control, depends strongly on the traffic scenario, the traffic network itself, and the simulation period.

Finally, it should be noted that a whole range of other control actions is possible, for example: changing the number of departing trips, changing the departure time of drivers (i.e., leave earlier or arrive later), influencing the drivers' route choice using dynamic route information panels (DRIPs) and variable message signs (VMSs), using public transportation, car pooling, ... For practical reasons however, we limit ourselves to the use of ramp metering in this project.

1.2 Available traffic data collection systems

During the second half of 2002, the project's consortium started negotiations with Flanders' Traffic Centre (located in Wilrijk, Antwerpen), in order to obtain a database containing the traffic measurements. In february 2003, these negotiations resulted in the availability of such a database for Flanders' highway road network [67]. All measurements are recorded by some 1655 sensors (i.e., approximately 200 cameras and 1500 single loop detectors), during the year 2001. These sensors are typically located right before and after a highway complex (i.e., an on-/off-ramp), with one sensor for each lane. Every minute, a sensor aggregates the following macroscopic measurements:

- the total number of passing vehicles,
- the total number of passing trucks,
- the average speed of these vehicles,
- and the average occupancy of the sensor.

Note that the decision of whether or not a vehicle is classified as a vehicle or a truck is made by a hard-coded algorithm in the sensors themselves. In total, there are 1655 sensors x 365 days x 24 hours/day x 60 minutes/hour = 869,868,000; corresponding to some 3.24 GB.

In Flanders, there is one very interesting road stretch, located on the E17 (Gent-Antwerpen) near the Kennedytunnel (see Figure 1.2 and Figure 1.3); here, 15 cameras are placed approximately every 500 metres over a distance of some 8 kilometres. Considering the quality of the data, some rudimentary checks were performed. One of the main concerns is the fact that these cameras were only operational during the second half of the year 2001. Furthermore, half of these cameras malfunctioned in some way, resulting in faulty measurements. This lead to new negotiations that started in november 2003, with the goal to obtain traffic measurements of this road stretch for the years 2000, 2002, and 2003 (the current status is pending). Nevertheless, coping with faulty measurements is a major concern that needs to be tackled; we therefore refer to the work done in section 2.2.4, dealing with filtering and estimating traffic data.

1.3 Other kinds of (traffic) data

Besides the aforementioned traffic measurements, other kinds of data are also available, e.g., travel time measurements from probe vehicles, GSM/GPS combinations, weather data, ... Because of the rather large amount of time it takes in order to obtain auxiliary data (it took some seven months just to get

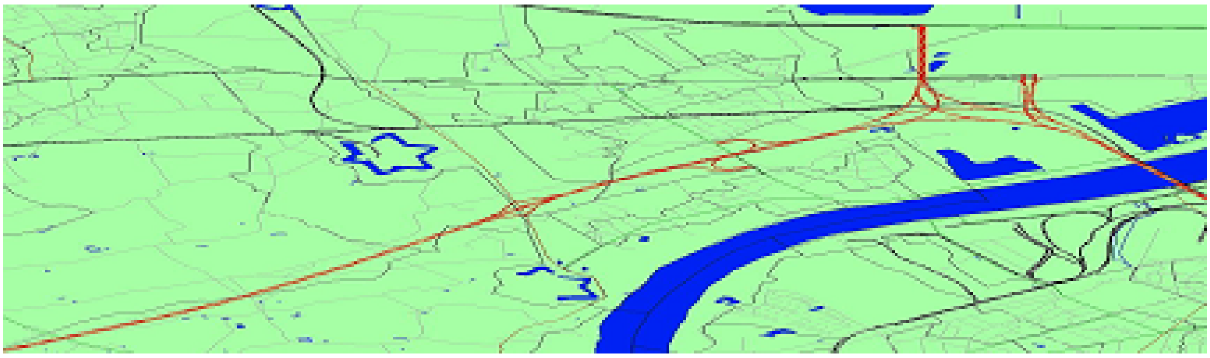


Figure 1.2: A stretch on the E17 (Gent-Antwerpen), densely populated with 15 cameras (CLO = camera Linkeroever); overview.

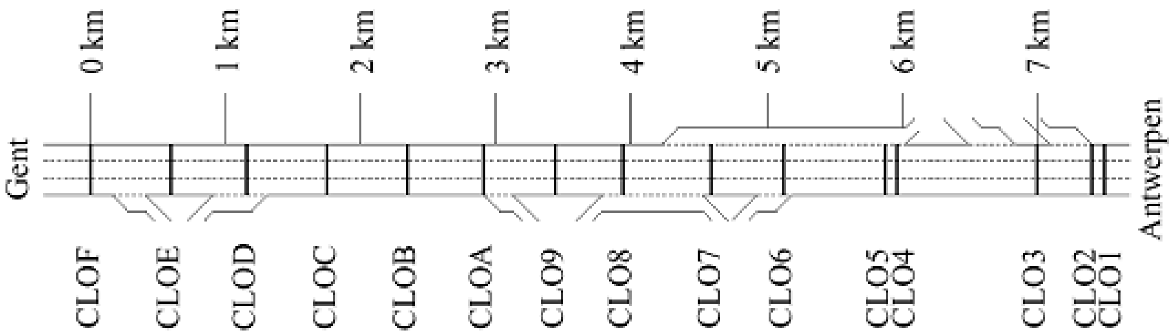


Figure 1.3: A stretch on the E17 (Gent-Antwerpen), densely populated with 15 cameras (CLO = camera Linkeroever); detailed view.

official approval from the governmental hierarchy), and because of certain privacy issues related to the use of GSM data, the project's consortium abandoned the use of this kind of data for the remainder of this project.

Chapter 2

Modelling

Within this part, the use of traffic flow models is dealt with. In a first section, a short overview of transportation models with their utilities, purposes, assumptions and properties will be set up. Secondly two types of dynamic traffic flow models that will be used in the project will be treated. At last a short description of an extended dynamic macroscopic traffic flow model will be given.

2.1 Transportation models

When some properties of our transportation system are described by a set of mathematically formulated assumptions, a transportation model is born. Because there is a wide range of intended properties to describe, there exist a lot of transportation models. This set of transportation models can be classified in several ways. Within this discussion we will distinguish these models on the basis of the features of the transportation system they intend to capture.

Roughly the decision processes of travellers within the transportation system can be clarified with the help of Figure 2.1.

We distinguish five types of sub-models that are closely related to the described travel options:

- Models that focus on when and where people want to travel are grouped in generation models. These models describe the complex decision processes that lead to the making of trips.
- Models that predict the destination of the trips are grouped within the distribution models. The distinction with the generation models can seem somewhat artificial. The decision of making a trip is often closely related to the choice of the destination of these trips.
- Within the proposed classification, the choice of transportation mode and vehicle type comes next.
- Within an assignment model, the route choice of all travellers is considered together. Given the amount of trips between several origins and destinations, and knowing the departure time and the modal choice of all travellers, an assignment model calculates the used routes and the resulting flows over a transportation network.
- A traffic operation model focuses on the driving behaviour of travellers. This detailed model is closely related with the observed traffic flow variables at road level.

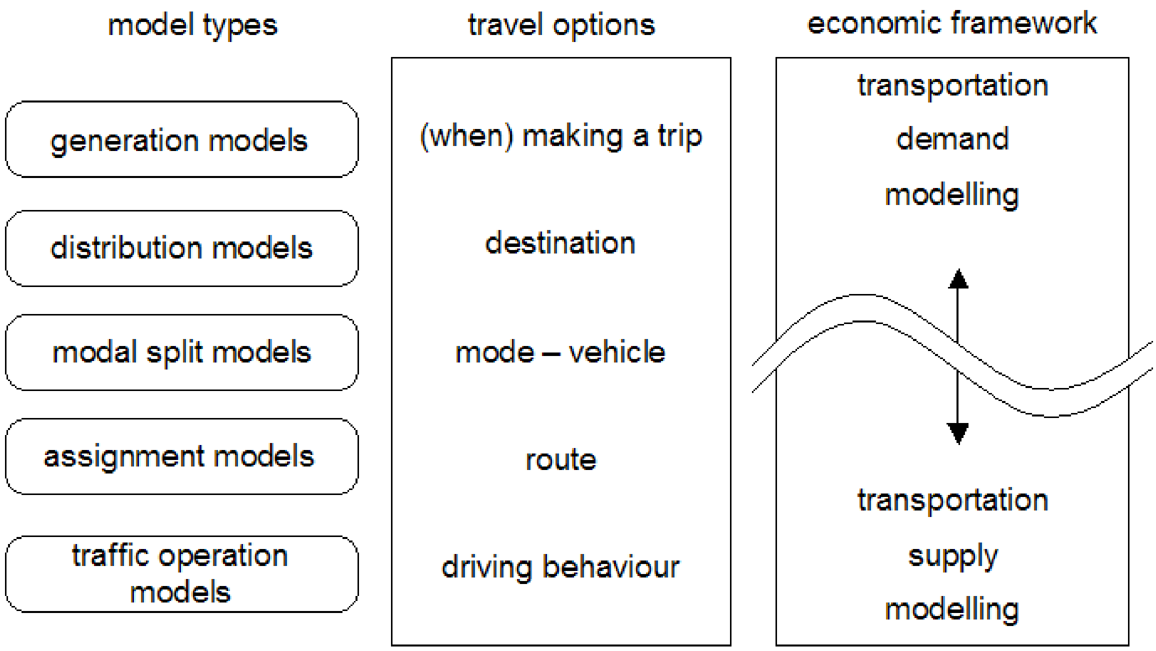


Figure 2.1: Building blocks of the transportation system.

When a particular transportation model is set up, some of these five sub-models are worked out. Within an economic framework we can state that every transportation model tries to describe an equilibrium between demand and supply. Depending on the considered travel options in the transportation system we can define a part as the demand modelling and a part as supply modelling . When considering an assignment model, the modal split will be considered as a result of the traffic demand. When setting up a generation model, the properties of the several transportation modes can be classified as transportation supply.

Traditionally, transportation economists focus on the generation, distribution and modal split sub-models. They are interesting in the results of the decision processes underlying the making of trips, the destination of them, the time of leaving and the transportation mode. Transportation engineers mostly work on the route choice and traffic flow operations.

Both economists and engineers have the purpose to improve the working of the transportation system. Transportation economists are searching to close the gap between the experienced and the real supply costs that travellers make. Transportation engineers' objective is to control traffic operations and route choice. The underlying idea for that, matches the economists viewpoint : there exists a difference between a user optimum and a system optimum.

Within this project the concepts of transportation economists will be projected on the traffic operation level. With the help of this, the control of traffic operations can be based on a complete and sustainable system optimum. Therefore an accurate mathematical description of traffic operations becomes necessary, justifying the use of dynamic traffic operation models.

2.2 Traffic flow models

Because traffic operations on the highway network are the main research area of this project, we can focus on traffic flow models. Different types exist that can describe the present traffic operations and can predict the effect of control measures. Based on the way vehicles on the highway are considered, we distinguish tree types of traffic flow models :

Microscopic

Microscopic models consider each vehicle separately. Within these simulation models the driving decisions of each driver are considered at every time step. This process comprehends effects of the vehicle properties, the infrastructure lay-out and the interaction with other vehicles.

Mesoscopic

Mesoscopic models are based on gas models. The vehicle stream is then seen as a stream of molecules. The underlying assumptions are also based on the interaction and moving properties of molecules. The difference between vehicles and molecules is that the amount of particles related to the dimensions of the encapsulated 'tube' or 'road', is much larger for gas models then for traffic.

Macroscopic

Macroscopic models describe traffic as a homogeneous fluid in a tube. Simplifications are made about acceleration and the speed is assumed to be a function of density.

Within our project two types of models will be combined:

- Macroscopic models can calculate in a fast way the state of traffic operation on a road. Therefore, this type of model will be used in real-time model based control strategies. Because of the simplifications made in these models, the effect of a control measure on a detailed scale is needed as input and not as result within these models.
- Microscopic models will be used in off-line calculations to predict the detailed effects of some new control measures. The results of these off-line simulations can lead to new parameters (e.g. a modified fundamental diagram) that will be used as input in the on-line macroscopic model. The microscopic model will also function as a mirror of the real world when testing the on-line macroscopic-model based control.

Within the CP/40 project, different traffic flow models are used and developed, depending on the specific role they play. At this moment, we are focussing on the following four types of models in our research:

- the first-order heterogeneous macroscopic LWR model,
- the second-order homogeneous macroscopic METANET model,
- the paradigm of microscopic traffic cellular automata (TCA) models,
- and the use of statistical models.

The first two models that we discuss in sections 2.2.1 and 2.2.2, are macroscopic continuum models; they provide a coarse description of traffic flows, in terms of flows, densities and average speeds. Because the goal of this project is to control traffic flows with respect to a certain cost criterion (see chapter 5 for more details), these macroscopic models are ideally suited for our control purposes.

As opposed to these macroscopic models, the class of TCA models, discussed in section 2.2.3, explicitly describes the dynamics of and interactions between individual vehicles. Because they are based on a very efficient modelling scheme, they are suitable candidates for using microscopic models when detailed calculations are necessary (e.g., computing noise pollution estimates).

Finally, when dealing with faulty detector measurements, as discussed at the end of section (1.2, we provide several statistical models that can generate good estimates of traffic conditions, based on noisy input data.

2.2.1 First-order heterogeneous macroscopic LWR model

In the original first order macroscopic traffic flow model, traffic on a highway is idealised to a homogeneous compressible fluid [42, 45]. As an important extension, we took the heterogeneous properties of traffic into account [52, 54]. To this end, the traffic flow was subdivided into homogeneous classes: each class consists of vehicles and drivers that share the same characteristics. Modelling heterogeneous traffic, in this case, comprises the description of homogeneous classes and the interactions between the different classes. These interactions are based on the user optimum:

- it is assumed that each driver maximises his own speed,
- and that fast vehicles are unable to affect the speed of slow vehicles (i.e., anisotropy).

For each road section, a class is characterised by its maximum speed, its (average) vehicle length and its capacity. The capacity pertaining to a class signifies the maximum traffic intensity that prevails when vehicles from that specific class have exclusive use of the road.

Note that this type of heterogeneous first-order traffic flow model has already been successfully used in section 1.1, when calculating the effects of an overtaking prohibition for trucks as a possible traffic control measure.

In a subsequent step, we extend the developed model for use on complete transport networks.

2.2.2 Second-order homogeneous macroscopic METANET model

When controlling a traffic flow, there is a need for a fast model that forms the core of a model based predictive controller (i.e., MPC). When testing the ramp metering setup, we choose the non-stochastic second-order METANET model [3]. With respect to the eternal debate of using first-order models on the one hand, and higher-order models on the other hand, we refer to the discussions in [10, 36, 13, 26, 1, 25]. Our choice is based on a trade-off between the model's accuracy and the computational complexity of its implementation. Note however, that the choice of the model class is not imposed by the used model predictive control (MPC) framework !

In the adopted METANET model, homogeneous traffic is modelled using aggregated traffic quantities; densities and flows are propagated through a network of links and nodes. The model has specific equations that allow description of weaving and merging effects at on-ramps and diverges. With respect to the MPC algorithm, the model includes an equation that explicitly takes the ramp metering rate into account (which is needed when calculating the effects of a certain control strategy using a model based predictive controller).

2.2.3 Microscopic traffic cellular automata (TCA) models

Besides the use of the classic macroscopic traffic flow models, we also investigated the class of traffic cellular automata (TCA) models [63, 60, 62]. These models start from a different point of view: by locally modelling the interactions between vehicles (as is common in microscopic traffic flow models), several emergent macroscopic effects arise. Although most microscopic models are computationally very complex, these TCA models perform a fine grained discretisation of space and time: whereas a macroscopic model typically divides the road in sections of approximately 500 metres (note that this can be arbitrarily chosen), roads in TCA models are divided in cells of 7.5 metres length (timesteps are of the order of 1 second) [57]. Based on this efficient modelling scheme, it is possible to perform detailed calculations (albeit with a certain upper level of accuracy due to the discretisation) for computing noise pollution estimates, et cetera, as used in the calculation of the sustainable cost function in section 5 [61].

One of the major goals is to allow the simulation of large scale networks (e.g., the whole of Flanders' highway road network) using these TCA models. As a possible approach for this task, we are currently focussing on the use of distributed computing whereby the computational load of the different roads is distributed over a set of networked computers/processors [56, 58, 59].

2.2.4 Statistical models

The goal of the contribution of the SYSTeMS group in the CP/40 project during 2003, was to develop practically useful (i.e., computationally simple, robust, and easy to use) estimation algorithms that use noisy, unreliable measurements coming from sensors (magnetic inductive single loop detectors and/or video cameras) as input. These sensors are typically located at a few isolated measurement points along the road, providing every minute the following available data (see also section 1.2 for more details): the number of cars that have passed the measurement location (often one sensor for each lane, and for two vehicle classes), and the average speed of these vehicles; these data contain measurement errors, and are often missing (at random).

The developed estimation algorithms generate good (i.e., minimum mean square error) estimates of the traffic conditions as output. These traffic conditions are described by the state variables:

- the traffic density (i.e., the number of vehicles per section),
- the average speed of the vehicles in a section, in kilometres/hour,
- and the traffic flow (i.e., the number of vehicles crossing the sensor location during consecutive one-minute intervals).

This data is provided at many locations along the road (not just the measurement locations) and at all times (not just the one-minute sampling time of the measurements).

As the most likely candidate for a good estimation procedure we selected recursive Bayesian filters, in which state updates are implemented via simulation (particle filters). In order to properly design these filters we developed the following two models:

- A new stochastic, hybrid model of road traffic evolution. The model was selected on the basis that it should be easy to use in the particle filter developed later, and so that it represented all the measurement data available in a simple relation to the traffic data to be estimated. The model is very easy to tune, requiring only a few parameters, such that each parameter has a clear physical meaning. The model was successfully validated by comparison with real data from the E17 (Gent-Antwerpen) road stretch (see Fig. 2), and by comparison with simulation results obtained via a thoroughly validated METANET simulation model. For more details, we refer to [49, 47, 48].

- A model of the measurement errors, both for the average speed and the number of vehicles crossing the sensor location (based on comparisons between the output of the measurement equipment and the human viewing of traffic video tapes).

These models were used in the development of a particle filter that evaluates the conditional density of the number of vehicles per section and their average speed, this at consecutive times. In between points in time, when measurements become available, the conditional distributions are updated by simulating the physical model developed earlier. Whenever a measurement becomes available (whether from all the sensors at the same time, or only from some of the sensors), the conditional distributions are updated according to Bayes' rule, evaluating the posterior likelihood of each sample of the simulation run. Fore more details, we refer to [49, 47, 48].

Our tool provides us with an estimator for traffic density and average speed that is robust against missing data, model mismatch, ... These estimates can be used directly in feedback control actions of traffic (setting signals at metered on-ramps, adaptive route guidance, adaptive pricing, ...), or can be used for estimating the mode of operation of the traffic (free flowing, congested, synchronised, jammed). An extension to models including both urban and freeway traffic is under development.

Chapter 3

Dynamic OD-estimation

In this section, we first discuss the concept of calculating the routes that drivers take given a certain network demand. We conclude with a specification of the project's testbed: a description of the network and the associated demand. This testbed will be used by the consortium in the various stages of the project, allowing benchmarked comparisons of our research results.

3.1 Dynamic traffic assignment

Being able to model the behaviour of vehicles on road sections is not sufficient. Even if we know the origins and destinations of all these vehicles (the so-called origin-destination pairs), there is still one item missing to completely model traffic on a network: the routes they choose between their origins and destinations. Nonetheless, this choice determines the travel time, pollution, ... and thus the costs.

The route choice problem described above, is solved in a dynamic traffic assignment model (DTA). It is based on a user equilibrium (or user optimum), which means that no driver can lower his travel time by choosing another route at any time [30]. Starting from the OD-matrices (i.e., the demand) and the available network infrastructure (e.g., the supply that can change by an incident), the DTA procedure calculates the distribution of vehicles over the possible routes (see Figure 3.1).

Because the mentioned user optimum is solely based on the travel times, it only incorporates the travel costs of the users. There are however other costs that take their toll and that are not included in the model: costs caused by air pollution, noise pollution, ... We can also try to find an equilibrium that does incorporate these costs; such an equilibrium is called a system optimum. In such an optimum, no driver can lower the total cost, by choosing another route at any time; and this is exactly where the sustainable cost function fits in.

Such an optimum is obviously much more preferrable from a governmental, managerial point of view. ATMSs are capable of controlling traffic in such a way that this system optimum can be approached. The DTA model we are currently working on, is a conditio sine qua non to obtain valuable results.

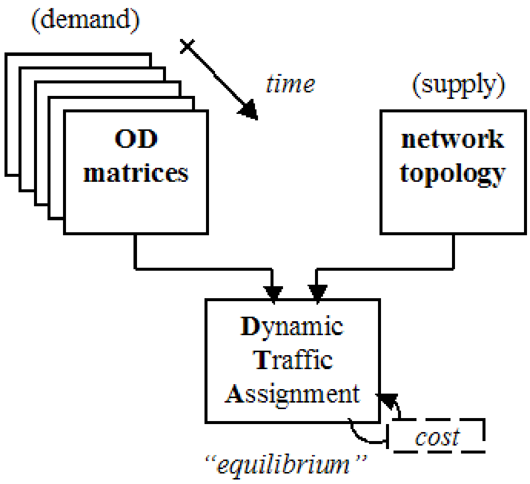


Figure 3.1: Dynamic traffic assignment combines the traffic load (i.e., the demand) and the network infrastructure with occurred incidents (i.e., the supply) to obtain an equilibrium.

Chapter 4

Model fitting and validation

In the CP/40 project, the model fitting is done using various methods, depending on the model used, and the approach taken. We first consider the techniques of using a spatio-temporal traffic filter and oblique N-curves in section 4.1, followed by the more classic approaches of least-squares regression in sections 4.2 and 4.3.

4.1 Calibration of the heterogeneous LWR model

The macroscopic first-order model discussed in section 2.2.1, is calibrated in a case study involving traffic measurements coming from ten double loop detectors at the Dutch A9 highway near Badhoevedorp (this highly qualitative data was obtained during the autumn of 1994). Data from October 20, 1994 was analysed using a spatio-temporal traffic filter to get a global view of the traffic pattern, and using the technique of oblique N-curves to obtain a more thorough picture of the congestion [52].

(I) A spatio-temporal traffic filter

The average speed and flow in a space-time diagram, are filtered using the filters from Treiber and Helbing [38]. The basic idea behind this filtering mechanism, is that the properties of the traffic flow move with the flow of traffic during free-flow conditions, whereas during congested conditions they travel against the direction of the flow.

The figures resulting from this filtering procedure are somewhat distorted, because it uses implicit properties of the LWR model with a triangular (density,flow) fundamental diagram.

(II) Oblique N-curves

By counting the number of vehicles that pass a given detector, we get the cumulative number of vehicles in function of the time. When these cumulative curves for the different detectors are represented in the same diagram, we can determine the number of vehicles and travel time between two detectors. Because these N-curves are non-decreasing, they reduce the readability of such a diagram. To compensate for this problem, Muñoz and Daganzo (based on the work of Cassidy and Windover), rescaled the N-curves in relation to a basic background flow [29]. Using this technique during calibration, allows for a minimisation of the difference between the measured and estimated N-curves (model parameters such as the free-flow speed for example, are determined for each class by averaging the measurements obtained from the individual double loop detectors). See Figure 4.1 for an example of these curves.

Validating the model is done using detector data based on a different traffic demand (using the parameters from the calibration phase); one of the main results is that the difference between observed and modelled data is strikingly minimal.

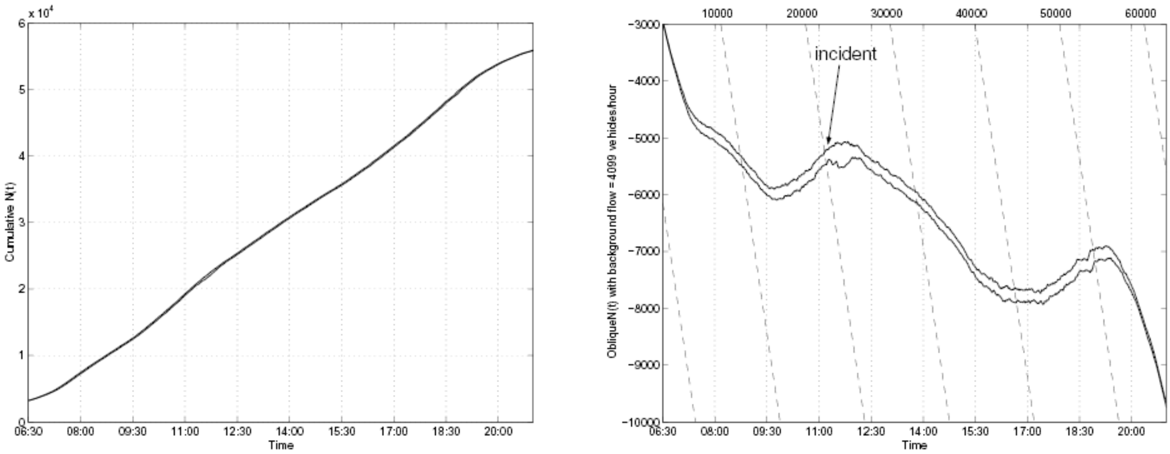


Figure 4.1: *Left:* a standard cumulative plot showing the number of passing vehicles at two detector locations; due to the graph's scale, both curves appear to lie on top of each other. *Right:* the same data but displayed using an oblique coordinate system, thereby enhancing the visibility (the dashed slanted lines have a slope corresponding to the subtracted background flow $q_b \approx 4100$ vehicles per hour). We can see a queue (probably caused due to an incident) growing at approximately 11:00, dissipating some time later at approximately 12:30. The shown detector data was taken from single inductive loop detectors, covering all three lanes of the E40 motorway between Erpe-Mere and Wetteren, Belgium. The shown data was recorded at Monday, April 4, 2003 (the detectors' sampling interval was one minute, the distance between the upstream and downstream detector stations was 8.1 kilometres).

4.2 Calibration of the METANET model

In order to calibrate the parameters of the macroscopic second-order model discussed in section 2.2.2, we collected them in a parameter vector; this allowed us to describe the parameter estimation problem as a non-linear least squares problem [66]. The data to calibrate the model, came from the detectors of the case study on the E17 road stretch (see section 1.2 for more details).

One of the main problems during this approach, is the fact that the resulting calibrated model isn't still always capable of mimicking the changes in traffic behaviour, due to external, non-modelled influences, and disturbances such as the weather, an obstruction on the highway, . . . To tackle this problem, an adaptive framework was developed that performs a refitting of the traffic model's parameters on a regular basis. This way, changes due to the non-modelled influences are tracked. By only updating the parameters with the highest sensitivity during the re-identification, the computational complexity of the re-identification process is kept as low as possible. This latter issue is very important if the identification is to be performed in an on-line setting.

Another extension to the modelling setup, was to include anticipatory effects due to the rerouting behaviour of drivers that react to a control strategy (this resulted in the use of an anticipative traffic assignment procedure that calculates the traffic load on the network, taking the time-dependent nature of the traffic demand into account).

4.3 Calibrations performed by CESAME

One of the project's partners, CESAME, pointed a smart subtlety out: when doing calibrations based on the (density,speed) and (density,flow) fundamental diagrams, it is important to know which parameters to calibrate from which diagrams. Measurements (provided by the MET) were obtained from ten loop detectors on the E411 (between Daussoulx and Overijse), over a period of 268 days between february 1st, and october 26th 2002. Based on this experimental data, the fundamental diagrams of Figure 4.2 were constructed.

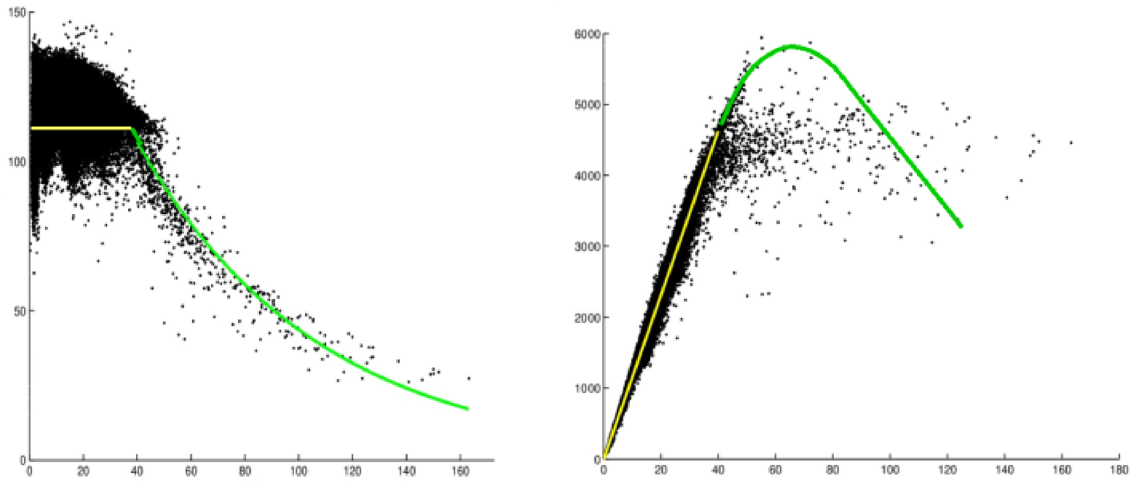


Figure 4.2: Two (density,speed) and (density,flow) fundamental diagrams for the E411.

As can be seen intuitively, when calibrating the free-flow speed, it is adviseable to use the (density,flow) fundamental diagram because the left branch exhibits a high (linear) correlation. In contrast to this, the scatter in the right part of this diagram is not suitable for calibration, leading us to the use of the (density,speed) fundamental diagram in which a better correlation between the datapoints is visible.

Chapter 5

Sustainable cost function

5.1 Introduction

Road travel involves not only time and resource costs for each road user but also congestion costs for other road users and environmental costs for the rest of society. Some of these costs are taken into account by the road user through the running costs of his vehicle, his private assessment of his time costs and insurance premiums. However, unless forced to do so by a specific policy measure, he does not consider the extra congestion costs to others or the pollution and noise damage or damage to the infrastructure his journey causes. Much literature in transport economics is devoted to the study of these external costs[1] and how different policies can be used to internalise them; usually via tolling regimes. See, for example, [46]. Such policy studies are mostly strategic and global, however, looking at the effect of policies at a national or international level over the long term and involving different transport modes.

In this study we wish to evaluate the effect of introducing advanced traffic management tools onto the existing road infrastructure in Belgium. Such tools include ramp metering, variable speed restrictions and dynamic route information panels. These types of control measures can be installed and operated at relatively low cost and can be adjusted to take account of changing traffic conditions. To determine which measures can best be used in a given traffic situation, we calculate their social cost from a sustainable cost function. The *sustainable cost function* (SCF) represents the total monetary cost of travel on the road network of interest and consists of time, air pollution, noise and accident cost components. Since implementing a particular measure (or combination of measures) affects the traffic flow variables, the components of the SCF must adequately reflect the relationship between these variables and the congestion and environmental costs. Hence in this study the research effort focuses on finding suitable functional forms as well as cost coefficients, which are appropriate for the Belgian road network.

The aim of this study is to compare traffic management tools in terms of their social costs for a given traffic demand and fixed road topology. We do not consider optimal policies to internalise external costs and it is therefore appropriate to use a total cost function. In the advanced traffic management system (ATMS), the sustainable cost function is combined with the dynamic traffic assignment model so that the SCF attains a global minimum over the road network. This is a complex iterative procedure and in the present study has only been applied to a limited, representative section of the Belgian road network. We also do not consider the ATMS in conjunction with regional or national policy measures, such as tolling. This could easily be done however.

[1] External costs are costs caused by one economic agent (firm, private consumer) that reduce the wellbeing of another agent without compensation.

In the following sections we briefly describe the road scheme to be modelled, the structure of the sustainable cost function and its implementation in the advanced traffic management system.

5.2 Sustainable traffic management systems

5.2.1 Transport model

When calculating transport costs from an economic point of view, there are a number of approaches available for modelling traffic flow. The choice of model depends on the aspects of the traffic situation which are of interest. For example, the effect of implementing regulatory policies on the transport network as a whole does not require detailed information on a road by road basis but allows for public/private road transport and non-road transport options (e.g. rail, ferry etc). Thus, in this case, an "aggregate" model is the most appropriate. For the present study however, we are interested in the effect of introducing traffic management schemes, such as ramp metering, speed restrictions and dynamic route information panels, on the existing road infrastructure. The traffic model is a dynamic, micro-simulation model. Hence speed (v), density (ρ) and flow (q) vary over a given link, in contrast to a static traffic model, for which these variables have constant values. Given this level of detail, it is not appropriate to calculate transport costs by aggregating all the traffic information onto one generalised "link". However, modelling the traffic network in this way means that we do not consider any possible shift of demand between modes. Table 5.1 summarises three basic types of traffic model and the travel options they incorporate.

	Travel options; choice of				
Model	*Mode*	*Departure time*	*Route*	*Vehicle*	*Driver behaviour*
Aggregate multi-modal models (e.g. TRENEN, typical strategic models)	yes	peak/off-peak only	no	yes	standard
Bottleneck models (e.g. METROPOLIS)	no	yes	yes	no	standard
Dynamic traffic assignment model (micro-simulation type)	no	no(*)	no(*)	no	individual driver response

Table 5.1: Transport Models; (*) Departure time and route have been fixed to simplify the model for this study (see below) but need not be in general.

In this study, the sustainable advanced traffic management system (ATMS) consists of a control loop, in which the controller searches for a new set-point at each time step. This is done in such a way that, given the O-D matrices, the sustainable cost function is minimised. Within the control loop, the dynamic traffic assignment (DTA) model assigns traffic to the road network. This is again an iterative process.

Given the complexity and iterative nature of the sustainable ATMS, modelling the complete Belgian road network or even some subsection of it is a complex task. A number of simplifications are therefore made for the purposes of this study. Firstly, the O-D matrices are assumed to be fixed so that the amount of traffic wishing to travel from a certain origin to a certain destination does not change when a control measure is implemented: road users cannot change their departure time. The O-D matrices are time dependent in that different matrices are used for different periods of the day. Secondly we consider a simplified road scheme which is considered to be broadly representative of the Belgian road network.

5.2.2 Description of the road network

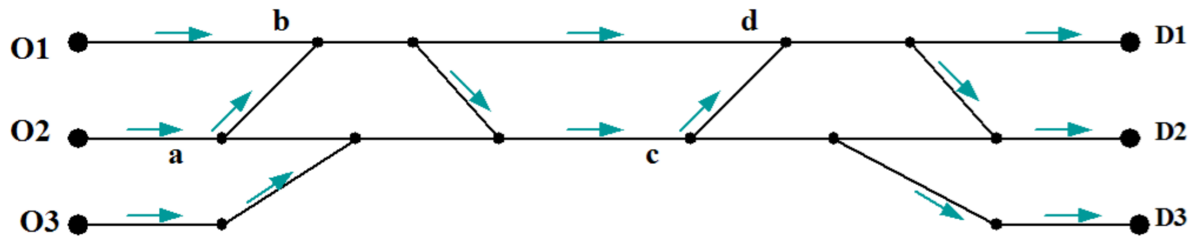


Figure 5.1: Simplified road network segment.

The road network segment consists of three uni-directional routes connecting three origins and three destinations, as shown above. The upper branch is a motorway (with three lanes and a speed limit of 120 km/h), the middle branch an 'urban' way (with two lanes and a speed limit of 90 km/h) and the lower branch a city road (with one lane and a speed limit of 70 km/h). These different road types have different pollution, accident and noise costs associated with them for a given set of traffic variables because of, among other things, the differences in housing density and driver behaviour. We clearly expect the traffic variables to play a role, however, with lower speeds on the narrower roads leading to further differences in congestion and environmental costs. The three roads are connected at a number of different points, allowing traffic to change the route they take to their destination, depending on the traffic conditions. Ramp metering can be implemented on the connecting roads a-b and c-d and speed restrictions enforced on the highway. It is also envisaged that dynamic traffic information panels could be installed on links O1-c and O2-a to temporarily divert vehicles from the motorway to the urban road or from the urban roads to the motorway when there is congestion.

Demand on the road network will vary over the day but this can be simply represented as a peak period when demand is high and an off-peak period. Differences in congestion and environmental costs between the two periods are mainly influenced by the differences in the traffic flow variables, which directly affect their travel time, emission, risk of an accident etc. In addition the value of time a road user associates with a journey increases when there is congestion.

The type of traffic using the road network (fleet mix) is also important for the calculation of environmental and congestion costs. For the calculation of congestion and environmental costs we assume a reasonably sophisticated fleet mix since journey type, vehicle type, fuel and emissions technology play a role here. This is explained in detail in the following section.

5.3 Sustainable cost function

The sustainable cost function (SCF) is a total cost function and represents the monetary valuation of changes in travel time, air pollution damage, noise damage, accident probability and infrastructure costs of a given equilibrium on the network. It contains both the internal costs (excluding taxes that are considered as transfers), which are taken account of by the vehicle user, and the external costs, which are paid for by society. An equilibrium is the result of a given traffic management scheme (ATMS) and a given traffic policy (in a broad sense: including tolls, taxes, capacity etc.). Thus, the effect of a given policy can be seen by comparing the SCF before and after the policy implementation. The global minimum of the SCF is obtained by iteratively selecting values of the policy variables and traffic arrangement, which minimise the SCF summed over all links on the given road network over the given time period.

Since the SCF is expressed in monetary terms, the calculation year can be important. The value of time associated with a journey, the cost per tonne of pollutants and the willingness to pay of users for vehicle safety, for example, may all vary over time. These changes can to some extent be accounted for by allowing for inflation and income effects[2]. In addition the fleet mix and risk of accidents may also change. Increased traffic demand and changes in vehicle technology can then be used to make new estimates. For the present study however, we are using a constructed demand and road network and comparing the effect of traffic management systems, which can be implemented relatively quickly on the existing network and do not involve long construction periods or high investment costs. Indeed the infrastructure and implementation costs have not been included.

In this case a precise definition of the calculation year is not necessary since the same data is used for all traffic management schemes and, for our constructed network, we are interested in the relative magnitude of the minimised SCF rather than absolute differences. In fact most of the data used in our calculations are for the period 1998 to 2002 and are implemented directly. Older data are adjusted for inflation.

Resource costs have been assumed to be the same for all scenarios. Producer surplus is also not considered and tax revenue would have to be added to the SCF in the case of tolling or taxes if these were dependent on the traffic conditions.

For clarity, only steady-state hourly costs are presented in the following sections. The coefficients and basic functional forms remain the same for transient traffic. Full details of their implementation can be found in Haut et al 2005.

5.3.1 Time costs

Time costs are a significant component of the SCF. The transport times are determined by the traffic model and are then multiplied by values of time (VOT) to give a monetary cost for the time period of interest. The VOT differ according to the nature of the journey (trip type), which depends on the type of vehicle and the reason for travelling. Thus, the total average travel time cost is defined as:

$$TTC = \sum_k q_k \sum_i \alpha_i (VOT_i^d t_d + VOT_i^e t_e + VOT_i^l t_l)$$

where: α_i is the proportion of trips of type i undertaken by traffic on path k, t_d is the travel time spent on path k (hours), q_k is the traffic flow on path k (vehicles/hour), VOT_i^d is the VOT for trip type i when travel time t_d is spent on path k (€/hour).

Similar definitions apply for early arrival (t_e) and late arrival (t_l) at the destination. Here, early and late are taken to be relative to the journey time under free flow conditions. Moreover, we assume that vehicles reach their destination immediately on leaving the network. This merely avoids the need to include travel costs for roads, which are not part of the modelled network. These costs are assumed to be identical for all equilibria. Since we have assumed that departure times are fixed, road users never arrive early at their destination; they arrive on time if there is no congestion and free flow conditions

[2] As individuals get richer they tend to value their time more and also the value of life and disability: this is called and income effect.

apply. Congestion costs can be considered to consist of schedule delay costs and queuing costs [4]. In this model, only queuing costs are captured The calculation of schedule delay costs would require a more sophisticated representation where departure time can vary. Queuing costs are a pure inefficiency and can be eliminated by tolling.

Journey types are important because they imply different VOT. Currently we distinguish passenger cars (differentiated by motives such as commuting, business and leisure), trucks and buses. [27] provide a detailed breakdown of daily household trips in Belgium, which are aggregated to fit the journey types for which VOT are available. The proportion of HGV journeys is determined from the 2000 vehicle fleet mix for Belgium [24]. The proportion of trips of each type is assumed to be the same in the peak and off-peak periods since more detailed data was not available. However, the VOT differ between congested and non-congested periods. The data used in the study are presented in Table 5.2 below.

	Trip type			
	Commuting	*Business*	*Leisure*	*HGV*
α	0.48	0.03	0.43	0.06
VOT (€/hour)	6	21	4	40
Congested VOT (€/hour)	9	31.5	6	40

Table 5.2: Time cost parameters.

The above VOT data were taken from the UNITE study [43], which aims to provide European standards for transport cost data, but it should be noted that other sources present different values. A study by [39] for example, suggests that the VOT for business should equal the wage. Income is not taken into account in the above calculations.

5.3.2 Air pollution costs

With the exception of greenhouse gases, air pollution effects are in general site specific. The best approach is to use emissions data and local meteorology in a dispersion model. An exposure-response curve is then used to determine the number of deaths, days of work lost etc., which all have an associated cost. This procedure is itself expensive and time consuming and, in any case, is not suitable for the 'representative' Belgian road network being used in this study.

In our approach we first specify the proportion of each vehicle type on the network according to the Belgian fleet mix [24]. The vehicle types include diesel and petrol cars, light and heavy goods vehicles and buses conforming to different EU standards, mainly depending on vehicle age. Then, for each vehicle type, we specify the speed related emission factor (g/km) for each of the pollutants carbon monoxide, nitrogen oxides, hydrocarbons, particulate matter, benzene and carbon dioxide[3]. These data are taken from the UK National Atmospheric Emissions Inventory[4]. Next, traffic flow variables are used to calculate total emissions (in tonnes) of each pollutant on a given link of the road network. These values can then be multiplied by existing euro/tonne cost estimates for different road types in Belgium [40] to obtain total costs per link, which are summed over the road network. Mayeres et al undertook an extensive study of the environmental costs of transport in Belgium. A detailed inventory was compiled based on life-cycle analysis and emission modelling of transport activities. The associated environmental damage was assessed using the 'bottom-up' approach of the EXTERNE project [6]. Hence average aggregate values could be obtained for the different road types.

The average air pollution cost per hour has the form:

[3]Poly-aromatic hydrocarbons, ozone and sulphur oxides are excluded due to lack of data

[4]NETCEN (2003): National Atmospheric Emissions Inventory – Vehicle emission factor database.

$$TPC = \sum_j \sum_p \sum_i (\beta_i F_{ip}(v_j)) C_p q_j d_j$$

where β_i is the proportion of vehicles of type i, F_{ip} is the emission factor (tonne/vehicle-km) for pollutant p and vehicle type i (speed dependent), C_p the cost for emissions of pollutant p (€/tonne), v_j the average vehicle speed on link j (km/hour), and d_j the length of link j (km).

The aggregated costs per tonne of the different pollutants emitted by road vehicles (C_p) are shown in Table 5.3 below. The other parameters from the air pollution cost function (β_i and F_{ip}) are available in a spreadsheet from the authors.

	Pollutants						
C_p€/tonne	PM2.5	NO_X	CO	Benzene	1,3-butadiene	HC	CO_2
highway	214035	3096	2	1317	50423	780	16,4
urban-way	34861	3312	0	246	10317	780	16,4
city road	388460	0	3	2330	87361	780	16,4

Table 5.3: Aggregated cost of pollutant emissions for Belgium.

5.3.3 Accident costs

Accident costs can be considered in two parts: the economic cost of an accident and the number of accidents that are expected to occur. The economic costs can be divided into the vehicle user's willingness to pay (WTP) for safety (a), friends' and relatives' WTP for the safety of the vehicle user (b), and the costs to the rest of society (c). The level of these costs depends on the severity of the accident (usually categorised as fatal, severe and light). The WTP represents society's monetary evaluation of the utility loss from an accident and includes emotional aspects and loss of leisure as well as income loss. The costs to the rest of society include medical, police and accident clean up costs. Part or all of (a) and (c) may be internalised by the road user through his insurance payments[5]. This distinction is not relevant here as we only consider total costs.

Values for (a) and (c) have been taken from the RECORDIT study [65], which provides WTP values for fatalities for ten European countries. Severe and light accidents are assumed to be 13% and 1% of these values respectively. Belgium is not included in RECORDIT. However, using the benefit transfer principle [15] values for Belgium were obtained from data for the Netherlands. The WTP is scaled according the GDP per capita at purchasing power parity[6]. The second category, WTP of friends and relatives, is generally ignored as data tend to be unreliable. Hence no (b) values are included in this report.

A large literature exists for calculating the number of accidents, of which a significant part is econometric (see, for example, [12, 20]). In general, the accident rate can depend on a wide range of factors including traffic flow, speed, congestion, road type but also driver behaviour and weather. It is also expected to be a U-shaped function of speed, speed variance and flow. More single vehicle accidents occur at high speeds or low flow and more multi-vehicle collisions when there is congestion. A number of possible formulations have been suggested, the simplest being a dependence on traffic flow alone: $N = aq^\alpha$. [16] suggest $a = 1$ for low/moderate flows and $a > 1$ for high flows based on empirical results. Following [50], for a fixed traffic volume, $N = \sum_i \left(av_i^2 + bv_i + c_i + d\sum_{j\neq i} \frac{(v_i - v_j)^2}{I-1} \right)$ also

[5] See for example [34, 35, 41].

[6] Data provided by Eurostat Nowcast for 2002.

depends on vehicle speed and according to Belmont [64], the form for two vehicle accidents is given by $N = (a + bv)q^2$. In the context of the EU MASTERS project (Managing Speeds of Traffic on European Roads), [5] presents an empirical form for N, which, however, only applies to rural, single carriageway roads.

The problem remains that it can be difficult to test or apply theoretical forms or to generalise from empirical studies, which tend to be site specific or use data aggregated over long time periods or different road types. We have therefore chosen a fairly simple approach.

Total accident costs are calculated according to the formula [8, 35]:

$$TAC = \sum_j \sum_{k,n} (a^n + b^n + c^n) \sum_i \gamma_k r^n_{ik} q_j d_j$$

where r_{ik} is the accident risk involving road user type k and another user type i on link type j, γ_k is the proportion of vehicle type k[7], and n denotes severity of the accident (fatal, severe, minor).

Additional traffic flow dependence is introduced via the accident risk, where, typically, we expect accident risk to be a function of the traffic variables and the road type. Given the information currently available, the new accident risk following a change of speed is determined from [19]:

$$r^n_{ik} = \frac{acc^{n*}_{ik}}{V^*_k} \frac{v}{v^*}^{x_n}$$

where[8] acc^{n*}_{ik} is the number of accidents of type n between user type i and user type k on link type j in reference flow, V^*_k is the number of vehicle-km driven by less protected road user k in the reference flow on link type j, v is the average speed in flow (km/h) on link type j, v^* is the average speed in reference flow (km/h) on link type j, and x_n is a parameter value depending on the accident type.

The risk of an accident of type n in the reference flow (on a given road type) is simply calculated from the observed annual number of accidents divided by the total number of vehicle-km driven on that road type by the accident victim. Annual data from Belgian road network[9] has been used for the reference flow. The average speed in the reference flow is set equal to the speed limit.

The coefficients for the accident costs calculations are shown in Table 5.4 and Table 5.5[10].

	v^*	acc^{f*}	acc^{s*}	acc^{l*}	V^*	x^f	x^s	x^l	γ
car-car	120	21	137	1199	2,57E+10	4	3	2	0,94
car-bus	120	2	9	41	2,57E+10	4	3	2	0,94
car-HGV	120	18	127	686	2,57E+10	4	3	2	0,94
bus-bus	120	0	0	0	2,24E+08	4	3	2	0
HGV-HGV	120	2	32	124	4,27E+09	4	3	2	0,06
bus-HGV	120	1	4	21	4,27E+09	4	3	2	0,06

Table 5.4: Coefficients for accident risk calculation.

[7] Here type refers to passenger car, HGV, bus only.
[8] Subscripts denoting link type (j) have been omitted for simplicity.
[9] Data for 2001 provided by the National Statistic Office.
[10] Data was unavailable for other road types at the time of this study. Highway data was therefore used for all road types but with the appropriate speed limit for each road type.

a			c		
fatal	severe	light	fatal	severe	light
1448285	188277	14483	144828	21931	2431

Table 5.5: Coefficients for accident cost calculations (€/accident).

5.3.4 Noise costs

As with air pollution, noise pollution effects are local and depend on the background noise level, which is a function of the local environment. It would be preferable to have detailed calculations using a noise dispersion model but this was not possible within the scope of this study. There are a number of different measures for quantifying noise exposure (the energy-equivalent level the 10 percentile level and the maximum level) and a large number of road traffic noise prediction models [51]. The approach taken here is to calculate noise exposure using the standard energy-equivalent measure, L_{Asq} (in dB), according to a general formula such that L_{Asq} depends on the traffic flow variables and housing density, which differ between our various road types. Noise disturbance above a given threshold can then be determined and, given the cost of one dB of noise disturbance, the total cost of noise pollution can be obtained. In line with the report by Robertson and co-authors, we have used the Nordic model formulae for the L_{AE} calculation.

$$
\begin{aligned}
L_{Asq} &= (73.5 + 25\log(\frac{v}{50})) \quad \text{for } v \geq 40 \text{ km/h for light vehicles and cars} \\
L_{Asq} &= 71.1 \quad \text{for } v < 40 \text{ km/h for light vehicles and cars} \\
L_{Asq} &= (80.5 + 30\log(\frac{v}{50})) \quad \text{for } 50 \leq v \leq 40 \text{ km/h} \\
L_{Asq} &= 80.5 \quad \text{for } v < 50 \text{ km/h}
\end{aligned}
$$

The total noise cost then takes the form:

$$TNC = C\sum_j \gamma_j(L_{Asq}(v_j) - L_{ref})n_j q_j d_j$$

where L_{ref} is the noise disturbance threshold over a given time period (dB), n_j is the housing density for road-type of link j (no/km), and C is the cost (€/dB/hh during one hour).

A value of 50dB was assumed for the noise threshold in agreement with Mayeres and van Dender 2001 and Navrud 2002, although this value is still somewhat subjective.

Exposure to noise can lead to health effects such as increased risk of cardio-vascular diseases, sleep disturbance and annoyance. Medical costs and productivity losses may be incurred but the main economic cost is the disutility associated with discomfort, reduced enjoyment of leisure activities, etc. A number of methods are available for the estimation of WTP to avoid noise damage per dB per household for the required time period (c). A hedonic pricing approach has already been used in a study for Belgium [41]. Stated preference values for WTP to avoid noise have been derived for several other European countries in a number of studies, which are summarised in Navrud 2002. The benefit transfer approach could be used to apply the WTP values to Belgium. However, these values vary between 2 and 99 euros/dB/household/year. The UNITE project [7] presents data for the exposure-response methodology. In this case exposure-response functions are used to calculate the expected number of cases (or risk) of different medical conditions as a function of noise exposure. Functions are also available for noise annoyance [32]. Total costs are then calculated by multiplying the number of cases by the monetary cost per case, which are provided for Finland, Germany and Italy. Again, the benefit transfer approach could be applied for Belgium.

Initially, the hedonic pricing approach has been selected as data are available for Belgium. The basic premise of this method is that the value of a house is a function of a number of environmental attributes including noise, as well as its intrinsic characteristics. Hence, when buying or renting a property, individuals can select a property with similar attributes in a noisy or quiet neighbourhood. It is reasonable to expect that the house in the quiet neighbourhood will be more expensive and thus the housing market can be regarded as a surrogate market for noise. We follow the approach of Mayeres and van Dender and assume that, ceteris parabis, an increase in noise exposure by 1dB results in house price depreciation of 0.5%. Using the average house price for the year 2000 we calculate the depreciated cost over a lifetime of 50 years. The housing density calculation for the urban and city roads is based on population density data from Mayeres et al 2001. The resulting model coefficients are presented in Table 5.6. Note that value of 0.0037 is equivalent to an annual value of 32 Ă/dB/hh.

road-type	C (€/dB/hh)	n (/km)	y_l	y_h	L_{ref} (dB)
highway	0,0037	200	0,94	0,06	50
urbanway	0,0037	38	0,94	0,06	50
city road	0,0037	17	0,94	0,06	50

Table 5.6: Coefficients for noise pollution cost calculation.

5.3.5 Infrastructure costs

Infrastructure costs have not been calculated as the details of the different ATMS are not available. These are not likely to differ significantly between the types of schemes being considered (speed restrictions etc).

5.4 Implementation

Cost functions have been constructed for time (congestion) costs, air pollution costs, accident costs and noise costs, which depend on the dynamic traffic variables. As each function has a monetary value, they can simply be added to give the total SCF for the network over a given time period. Coefficients for these functions have been compiled and are presented in a spreadsheet, together with their functional forms, in order that they can be easily incorporated into the traffic model by the project partners[11].

The SCF has been incorporated into the traffic model by CESAME. This involved extensive collaboration between CESAME and ETE personnel, to correctly transfer the information from the SCF spreadsheet into the model framework. The results of this work are reported by CESAME [22]. They first derive a steady-state optimum and then look at the more complicated case of transient traffic. They show that time costs are the dominant component of the SCF and there is a trade off between pollution and time costs: minimising only pollution costs leads to a decrease in pollution costs but an increase in time costs compared with minimisation of the total SCF. The results of their research can be found in chapter 7.

[11] Available from the authors.

5.5 Conclusions

A method has been established for the evaluation of Advanced Traffic Management Systems (ATMS) using the sustainable cost function developed in this study. This function allows an ATMS to be appraised in terms of its congestion and environmental costs, which are calculated as monetary values. The approach differs from other economic transport modelling studies in that the time and environmental costs depend on traffic flow variables, which vary dynamically over the road network. The sustainable cost function is incorporated directly into the traffic modelling system so that the total social cost of travel on the road network is minimised instead of travel time only, as is often the case in transport management.

The sophistication of the sustainable cost function was limited by the resources available for the study. In addition, a specific geographical location was not modelled. Given these limitations, it was considered appropriate to use state-of-the-art values from the literature and data from detailed studies previously carried out for Belgium as input to the components of the SCF. The resulting functions and parameters would benefit from sensitivity analysis, as this process clearly required a number of simplifying assumptions to be made. Moreover, even when it is appropriate to use the benefit transfer principle to exploit data collected in a different location for the calculation of air pollution, accident and noise costs, there are some wide variations in the data reported in the literature. Indeed, for accident costs, there is also no consensus on the calculation method for accident risk.

Overall, the sustainable cost function appears to be an effective tool for evaluating traffic management schemes. It can be combined with the dynamic traffic assignment model in a relatively straightforward way. The effect of changing a policy measure due to changes in traffic conditions (adjusting speed restrictions or ramp metering, for example) can be readily seen and compared. Time costs clearly dominate environmental costs in the sustainable cost function. However, it is still potentially an interesting policy instrument as a policy maker could choose to implement traffic control measures which reduced environmental costs alone.

Chapter 6

Control techniques

We give an example of a dynamic traffic management (DTM) strategy for a highway: *ramp metering*. We begin with a description of the ramp metering setup after which we discuss the cost function that generally follows from the policy and practical issues. This cost function describes the quality of the realized traffic state. After this, we show how MPC based control leads to an optimisation problem with constraints; we illustrate this by means of simulation results.

6.1 Description of a ramp metering setup

Ramp metering is a DTM measure that consists of allowing vehicles to enter the highway by drops. Each highway has a critical traffic density, beyond which the traffic flow declines with increasing traffic density. The idea behind ramp metering is to keep the traffic flow of vehicles on the highway as optimal as possible, by keeping the traffic density always below the critical density. In practice, this can be realized by positioning a traffic light at the on ramp of a highway, this in order to limit the number of vehicles that is allowed to enter the highway. The control parameter which allows influencing the state of traffic on the highway is the number of vehicles that are allowed on the highway. Or in other words, the duration of the green phase of the traffic light. When a practical implementation of ramp metering is concerned, it is necessary to accomodate enough space on the on ramp in order to hold the waiting the queue of vehicles, without causing any hindrance to the underlying road network.

In Figure 6.1 we see a schematic representation of a highway with an on ramp. At this on ramp, a traffic light is placed and room is provided in order to accomodate a waiting queue.

A practical advantage of ramp metering is the simplicity with which it can be compelled. Drivers are obliged to stop in front of the red light, so as not to cause any heavy traffic infringment. It is thus supposed that no vehicle enters the highway during the red phase of the traffic light. Furthermore, it is easy to quantify the number of vehicles that enter the highway : by keeping the duration of the traffic light's green phase very small, we can dose the flow on a vehicle by vehicle basis.

6.2 Model predictive control

Model predictive control (MPC) is an on-line control technique that is used for optimal control of a ramp metering setup. MPC uses a model in order to predict the future evolution of the traffic state, given a known traffic demand. A control signal is searched for in order to minimize a predefined cost

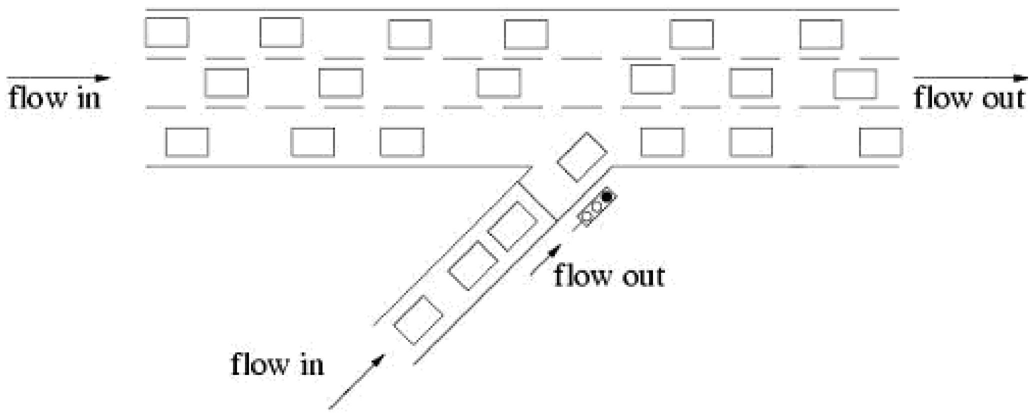


Figure 6.1: Schematic representation of a ramp metering setup : two consecutive highway sections and an on ramp with a traffic light and waiting queue.

function. MPC uses a rolling horizon, which means that a prediction horizon N_p is defined such that at each time instant k of the prediction horizon the N_p control signals can be calculated by minimizing the cost function over the prediction horizon. These control signals are the number of vehicles that are allow to enter the highway at time instant k. The value of the cost function over the prediction horizon is calculated by using a traffic state that is simulated by a traffic model. Here the METANET model, described by Papageorgiou, was chosen as a traffic model.

The calculation of the N_p control signals for the prediction horizon is rather computationally intensive, especially with an increasing size of the prediction horizon. Therefore, a control horizon N_c is defined as the period during which the control signals are allowed to vary. The length of the control horizon is smaller than or equal to the length of the prediction horizon. After the control horizon has passed, it is assumed that the control signal remains constant (see Figure 6.2). By the control horizon's definition, the number of parameters to optimize (and as such, the computational complexity) is reduced. In the context of a rolling horizon, only the first control signal of the prediction horizon is applied to the system. The other signals are discarded. The traffic state is then updated by means of measurements, the control and prediction horizons are advanced in time and the whole cycle starts over again. The parameters N_p and N_c are to be chosen such that a balance is found between on the one hand the complexity and on the other hand the accuracy of the controller.

6.3 The cost function

A governmental traffic policy might consist of keeping the flows on the highways at maximum capacity. We can translate this to the control of the ramp metering setup in the following way : the total time spent in the system has to be minimal. The total time spent is composed of the time spent by the vehicles on the highway and the vehicles in the waiting queue at the on ramp. This results in the following cost function :

$$J = \sum_{k=k_0}^{k_0+N_p-1} \left(\sum_{i=1}^{S} \rho_i(k) q_i n_i \Delta T + \sum_{j=1}^{R} w_j \Delta T \right) \tag{6.1}$$

In this formula, k_0 is the starting time, N_p is the prediction horizon, S is the number of sections on the highway, i is the section under consideration in the summation, $\rho_i(k)$ is the density on section i at

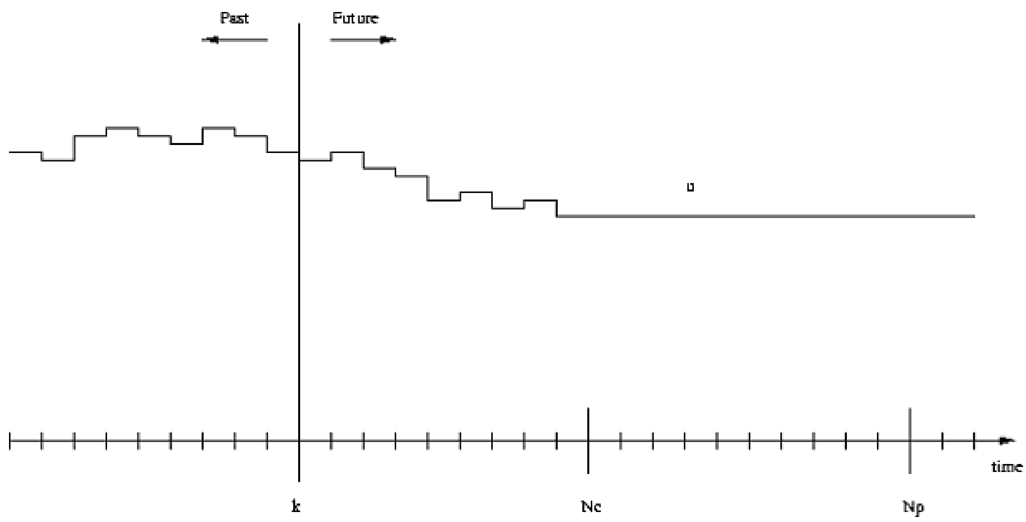


Figure 6.2: Schematic representation of the rolling horizon principle used in MPC. At timestep k we look with the model over a prediction horizon N_p and trie to establish a control signal that minimizes the cost function. The control signal itself is allowed to vary during timesteps k to $k + N_c$, after which it remains constant. The first step of the control signal is then applied, after which both horizons advance one timestep.

time k, q_i is the flow on section i at time k, n_i is the number of lanes on highway section i, ΔT is the time step with which the discrete controller is updated (typically 1 minute), R is the number of ramps, j is the ramp under consideration in the summation and w_j is the waiting time on the on-ramp.

6.4 An illustrative experiment

As an illustration, an experiment was performed based on a model of a section the E17 highway between Ghent and Antwerp. A ramp metering setup was simulated at the on ramp at Linkeroever (i.e., the first left on ramp). The total time spent during a simulated morning rush hour was compared with and without an MPC controlled setup. Figure 4 gives an overview of the simulated section of the highway. The total time spent in the network is used for the cost function. The prediction horizon N_p was taken to be 10. Because the sampling step of the controller was considered to be 1 minute (it's rather useless to adapt the metering rates more frequently), the controller only 'looks into the future' for about 10 minutes. In order to limit the needed computation power, the control horizon was chosen to be 5, or in other words, the control signal can vary during the first 5 minutes of a simulation period, after which it is considered to remain constant. A big advantage of MPC based control is the fact that certain constraints can be imposed to the optimisation. For example, it is assumed that the available space on the on ramp can only accomodate at most 100 vehicles. Besides the maximal length of the waiting queue, other constraints can be imposed.

The simulated morning rush hour runs from 5h until 10h. Figure 6.3 shows a summary of the simulation results. In the first plot, the course of the traffic demand on the on ramp is shown. It can be seen that this traffic demand increases until it decreased after the morning rus hour. The second plot shows the control paramter, which is the metering rate or the fraction of the total capacity of the on ramp which is allowed to enter the highway. We can see that the metering rate initially equals 1 and that all vehicles that want to enter the highway are allowed to do so (the length of the waiting queue is shown in the third plot and is zero). In the fourth plot we can see the course of the traffic density in time. The dotted line represents the critical density of the highway, or in other words : if this density is exceeded, the flow of the vehicles on the highway begins to decline with increasing density (i.e., congestion). We can see that because of the increased traffic demand, the density on the highway begins to increase. However, we traffic density on the highway tends to exceed the critical density, the ramp metering control is applied : the metering rate declines, reducing the inflow on the highway below the traffic demand, resulting in the formation of a waiting queue at the on ramp. The density on the highway fluctuates about the critical density (this is the density at which the flow on the highway is maximal). At a certain moment, the length of the waiting queue reaches its maximal value (100 vehicles) and the controller is necessitated to increase the metering rate, causing an increase of the density on the highway beyond the critical density. The accompanying average speed on the high way is shown in plot 5. After a while, traffic demand decreases, which results in a dissolvement of the waiting queue (this can be seen in the first plot, in which the real inflow of vehicles to the highway is greater than the demand).

If we look at the total time spent in the network (i.e., highway and waiting queue), we can see that, in case no ramp metering is applied, this is equal to 2960 vehicle-houres during the complete morning rush hour. However, when MPC based control is performed, the total time spent in the network is reduced to 2843 vehicle-hours, resulting in a reduction of 117 vehicle-hours (4%) during the morning rush hour (i.e., when ramp metering is active). While the queue on the on-ramp is building up, the morning rush hour jam is shorter in time than without ramp metering. Once the maximal queue length is reached, congestion sets in.

6.5 Conclusion

The setup of a model based dynamic traffic management (DTM) system starts with the definition of a policy. This policy is then translated into a cost function. An MPC based controller uses the concept of a rolling horizon. The control signals which lead to a minimum value of the cost function are searched by means of optimisation. In this process, a traffic model is used. Only the first control signal is applied to the system, after which the model is adapted to the changed traffic state on the highway and the horizons are advanced. MPC is capable of finding a solution which adheres to strict constraints (e.g., a maximal length of the waiting queue). In order to suppress the computational complexity, a control horizon is defined besides the prediction horizon.

In the presented example, a simple application of MPC based ramp metering on a single on ramp was shown. This resulted in a 4% reduction of the total time spent on the considered section of the E17 highway.

In addition, we also compared two main ramp metering algorithms in this project [66]:

- using the classic ALINEA controller logic,
- and using a model predictive control (MPC) framework.

ALINEA is a PID-like feedback control methodology that is widely in use today [37]. Here, the controller tries to maintain the density on the highway equal to some setpoint (which is typically a value lower than the critical density at which traffic breakdown occurs). The ALINEA controller steers the traffic density on the highway towards this preset density, varying the metering rate of the on-ramp and taking its maximal queue length into account.

Model predictive control (MPC) is a very popular controller design method in the process industry, and is suitable for control of traffic operations [66, 2].

The ALINEA-based ramp metering controller was used to act as a point of reference when assessing the performance of the MPC-based controller (which is not restricted to ramp metering control itself).

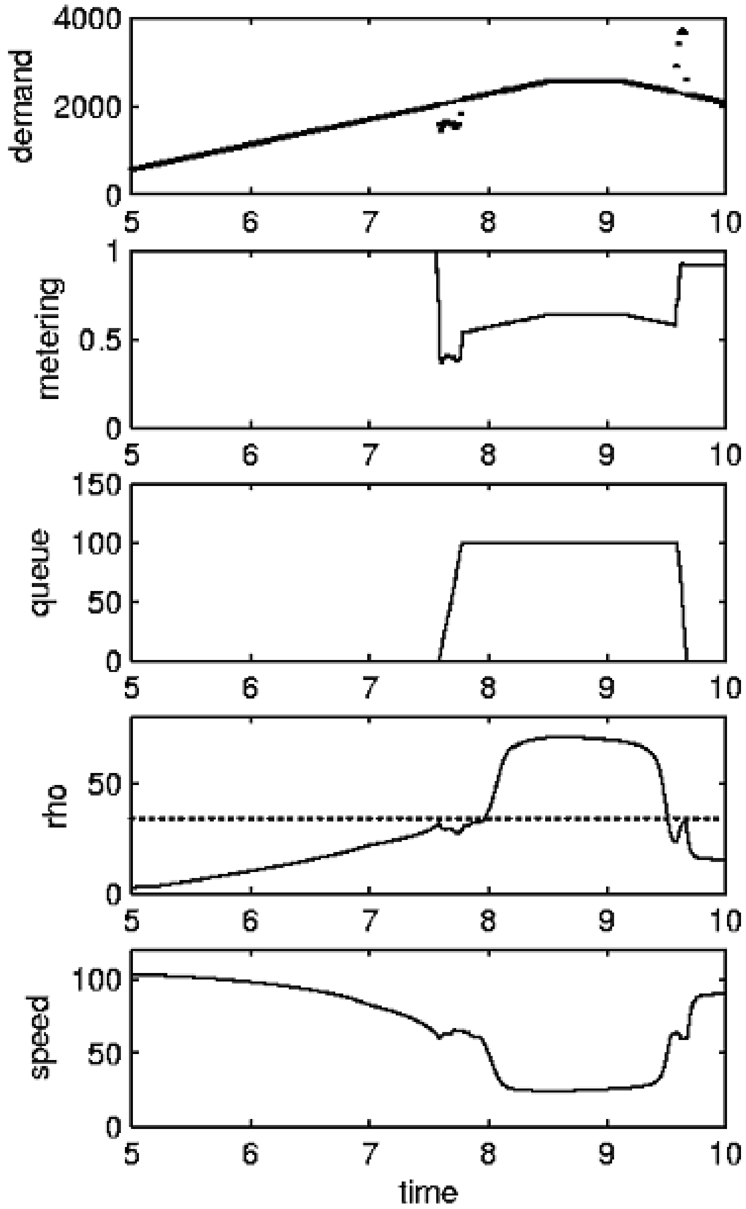


Figure 6.3: Overview of the simulation results for a morning rush hour and MPC based control at the E17 highway between Ghent and Antwerp.

The optimisation of a cost function in the MPC framework, makes it on the one hand very flexible, but on the other hand it increases the computational complexity.

Both of these control algorithms were considered in a non-coordinated and a coordinated setup.

Comparing the different scenarios (using the total-time-spent (TTS) and maximal queue length as performance indicators), the main results are [Bel03]:

- The ALINEA-based controllers result in a gain in the TTS during the simulated rush hour; the controllers are able to limit the queues approximately to the maximum queue length. A noticeable effect is the oscillating behaviour of the control signal, due to the constraints on the maximal queue length overriding the controller.

- The MPC-based controllers can realise a higher performance (i.e., a smaller TTS) than their ALINEA-counterparts. Moreover, their control signals are very smooth, even with the peak traffic demands during the rush hour.

With respect to the cases of coordinated and non-coordinated control, we can state the following results:

- Only the MPC-based controller was easily extended to handle coordinated control strategies.
- As expected, a non-coordinated MPC-control setup results in a worser performance than its coordinated counterpart. However, this is still better than an ALINEA-based control of several on-ramps simultaneously.

Chapter 7

Optimisation

7.1 Introduction

The research has focused on two main issues which are briefly reviewed in this introduction.

1. Traffic control in steady-state conditions: Development and validation of an optimisation procedure for the control of the traffic flow distribution in a complex network.

We consider a general complex network, with several origins and destinations and with several paths connecting each origin to each destination. Obviously there exist several traffic flow distributions satisfying the Origin/Destination (O/D) demand. The goal is to determine the steady state flow distribution which optimises a sustainability cost function taking into account economical aspects related to the network operation: time cost, pollution cost, accident cost, etc. The problem is addressed in the steady-state situation, i.e. for a constant demand and constant flow conditions on the arcs of the network. The solution allows to evaluate the benefit to be expected from traffic control actions which achieve the desired flow distribution (such as ramp metering imposing the value of the flows on the controlled arcs), as well as the performance degradation in case of non optimal flow distribution. The successive steps of this study are as follows:

1. Calibration of the flow density model on the basis of data provided by the MET (Walloon Region) for the E411 highway (Section 2).
2. Characterisation of the network topology achieved for both the CP/40 benchmark network and the E411-N4 section between Eghezée and Brussels (Section3).
3. Development of a procedure allowing to evaluate the sustainable cost relative to the steady-state operation of a given network (Section5). The econometric data relative to the operation costs (time costs, pollution costs, accident costs) have been provided by KUL/ETE, but it was necessary to aggregate this information in order to produce a global sustainability cost for a given network, expressed in euros per hour of operation of the network under given steady-state conditions.
4. Formulation of the optimisation problem. It is shown that the optimisation problem can be formulated as a standard linear programming problem, for flow maximisation (Section 4) as well as for the sustainability cost minimisation (Section 6).

Numerical results, relative to CP/40 and E411-N4 networks, for the maximum flow and for the minimum sustainable cost problems illustrate the approach. Specifically they allow to investigate the influence of control strategies by speed limitation on the traffic distribution and on the sustainability cost (Sections 4 and 6).

2. Traffic control in unsteady conditions: Development and validation of a traffic dynamical simulator the analysis of on-line traffic control in complex networks.

In the real world, the traffic situation on a network is often not in steady-state conditions. From a macroscopic point of view, the traffic evolution may then be described by a nonlinear Partial Differential Equations (PDE) model. We have extended the classical LWR model, as well as the corresponding numerical integration scheme, in order to take the network complexity into account. Several classes of vehicles, depending on the different paths between the origins and the destinations, have been introduced. Moreover, according to the network topology, new models have been derived for various types of network connection nodes such as origin, end, divergences, on-ramp, off-ramp, merge. In order to make the model more realistic, a special attention has been paid to the "capacity drop" phenomenon (Section 8). Furthermore, it was required also to generalise the sustainable cost evaluation procedure in order to take the unsteady-state effects into account (Section 7). The resulting dynamical simulation software developed in this project provides an efficient numerical integration of the model equations and allows to follow the evolution of the density on each arc, while evaluating in parallel the evolution of the sustainable cost rate. The report presents simulation results relative to the CP/40 benchmark and the E411-N4 network. The software appears to be a useful tool to investigate the efficiency of traffic control strategies for complex networks. In this report, it is used to study the effects of traffic management and control policies (including ramp metering and speed limitation) in order to avoid jam propagations along the highways, especially in presence of the capacity drop phenomenon at the junctions (Section 9).

7.2 Traffic variables

7.2.1 Definition

In the fluid flow paradigm for road traffic modelling, three basic variables are considered: the density ρ, the flow q and the speed u. At a time instant t and at point x along a road segment, the density $\rho(x,t)$ [veh/h] is the number of vehicles per unit of length, the flow $q(x,t)$ [veh/h] is the number of vehicles passing the point x per unit of time, the speed $u(x,t)$ [km/h] is the speed of the vehicles passing at point x at time t. Obviously, by definition, the flow is the product of the density by the speed which implies that the three variables are related by the following fundamental modelling relation:

$$q(x,t) = \rho(x,t)\ u(x,t) \tag{7.1}$$

The traffic analysis is then based on an additional modelling assumption regarding the relation between the speed and the density. When the traffic density is low, it is assumed that the vehicles move, in the average, at a constant speed which is close to the maximal admissible speed (denoted u_{max}). When the density exceeds a critical value ρ^*, it is then assumed that the interaction between the drivers become significant and the drivers reduce their speed as the density increases. Finally, when the traffic is totally congested, the maximal density ρ_{max} is reached and all the vehicles are stopped.

In this report, we consider three road types: highways, secondary (or national) roads and city roads. For each road type, the characteristic parameters are given in Table 7.1.

By the sake of simplicity and without a real loss of accuracy for our purpose, a piecewise linear relation is used to represent the relation $Q(\rho) = \rho U(\rho)$.

	u_{max} [km/h]	ρ_{max} [veh/km]	ρ^* [veh/km]	$q_c = u_{max}\rho^*$ [veh/h]
highway	120	270	50	6000
national road	90	180	44.4	4000
city road	70	90	28.6	2000

Table 7.1: Road type characteristic parameters.

$$Q(\rho) = \begin{cases} u_{max}\rho & \text{if } 0 \leq \rho \leq \rho^* \\ q_c \frac{\rho_{max}-\rho}{\rho_{max}-\rho^*} & \text{if } \rho^* \leq \rho \leq \rho_{max} \end{cases}$$

This implies that the $U(\rho)$ relation is as follows:

$$U(\rho) = \begin{cases} u_{max} & \text{if } 0 \leq \rho \leq \rho^* \\ \frac{q_c}{\rho} \frac{\rho_{max}-\rho}{\rho_{max}-\rho^*} & \text{if } \rho^* \leq \rho \leq \rho_{max} \end{cases}$$

These functions are illustrated in Figure 7.1 for the three road types.

In this report, we will investigate the influence of some speed limitations on the highway. The presence of speed limitations can be expressed as a modification of the speed-flow-density relation (see Fig. 7.2).

7.2.2 Experimental model calibration

The parameters of the model have been calibrated from traffic data provided by the MET (Ministère de l'Equipement et des Transports, Region Wallone) for the E411 highway.

Flow and speed measurements were provided by MET on an hourly basis, at ten stations distributed between Daussoulx and Overijse for a period of 268 days between February 1, 2002 and October 26, 2002 (see Fig.7.3). These data have been used to identify the parameters of the flow-density (Fig.7.4) and the speed-density (Fig.7.5) relations. The results are given in Tab.7.1 for highways. The parameters for secondary and city roads in Tab.7.1 were derived proportionally to the highway parameters.

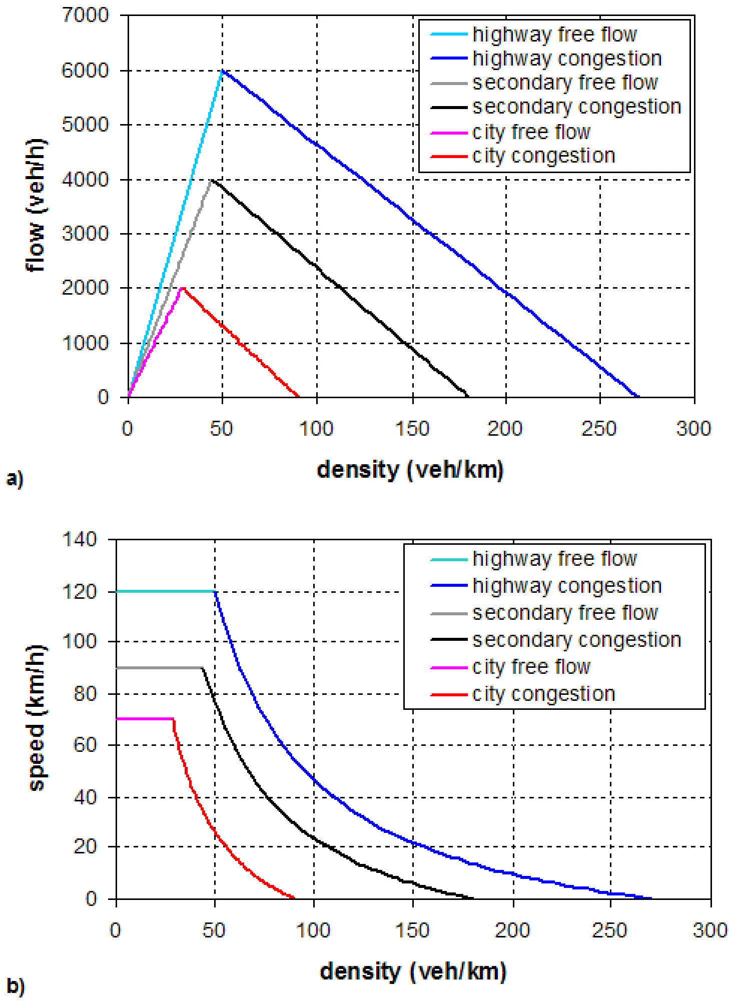


Figure 7.1: Speed-flow-density relations for highway ($u_{max} = 120km/h$), secondary road ($u_{max} = 90km/h$) and city road ($u_{max} = 70km/h$) a) q vs ρ b) u vs ρ.

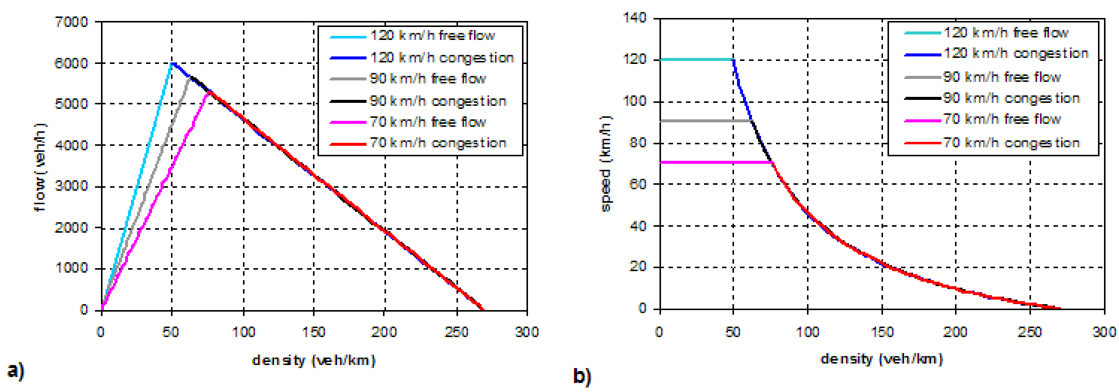


Figure 7.2: The influence of speed limitations on the speed-flow-density relations .

Figure 7.3: E411 section.

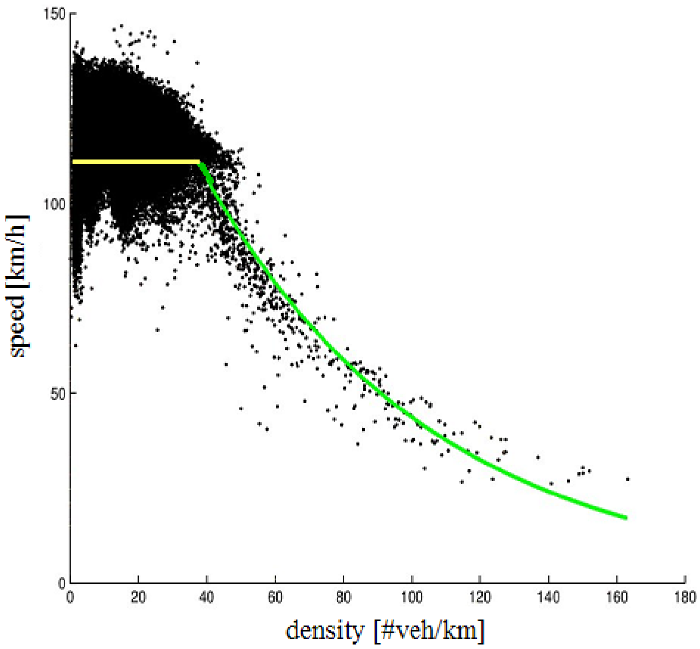


Figure 7.4: The experimental speed-density relation.

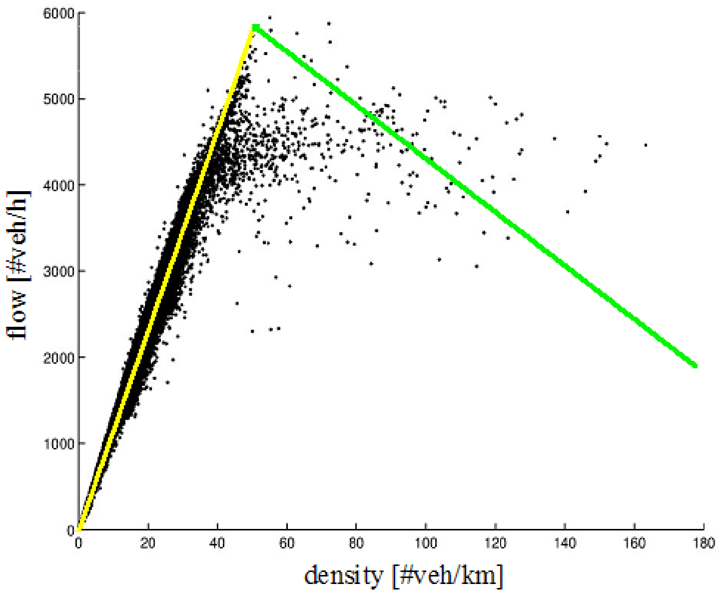


Figure 7.5: The experimental flow-density relation.

7.3 Networks

7.3.1 Definition

A traffic network is described as a directed graph $G = (N, A)$ composed of a set of nodes N connected by a set of arcs A. The nodes of the graph represent the road junctions in the network. In addition, a network has some origin nodes $(O \subset N)$ and some destinations nodes $(D \subset N)$.

The arcs of the graph represent the road segments between the junctions. An arc connecting a node $i \in N$ to a node $j \in N$ is denoted $(i, j) \in A$. The set of arcs which exit from a node i is denoted as A_i^- and the set of arcs which enter a node i is denoted A_i^+.

A path k is a sequence of arcs that are travelled between an origin $r \in O$ and a destination $s \in D$. A graph can possibly have different paths k for the same OD pair. The set of paths between an origin $r \in O$ and a destination $s \in D$ is denoted K_{rs}.

Each arc corresponds to a uniform road segment characterised by its length and its speed limit. In the present project, there are three different road types: highway or freeway , secondary or national road and city road.

7.3.2 DWTC-CP/40 project network

For the DWTC-CP/40 project, an oriented road network is proposed, see Fig.7.6. There are three origins (O1, O2 and O3) and three destinations (D1, D2 and D3). The Upper branch, U represents a highway with three lanes and with a maximum speed of 120 km/h. The Middle branch, M represents a secondary road of two lanes with a maximum speed of 90 km/h. The Lower branch,L is a single lane road with a maximum speed of 70km/h. Four road segments connect them together (ac, ef, gh, jk). The OD matrix is presented as a proportion of origin demand for each destination for each pair OD, see Table 7.2. The lengths of the road segments represented by the graph are given in Table 7.3. The different paths between the origins and the destinations are given in Table 7.4. They are numbered from 1 to 18 and have also a colour code shown in Fig. 7.7

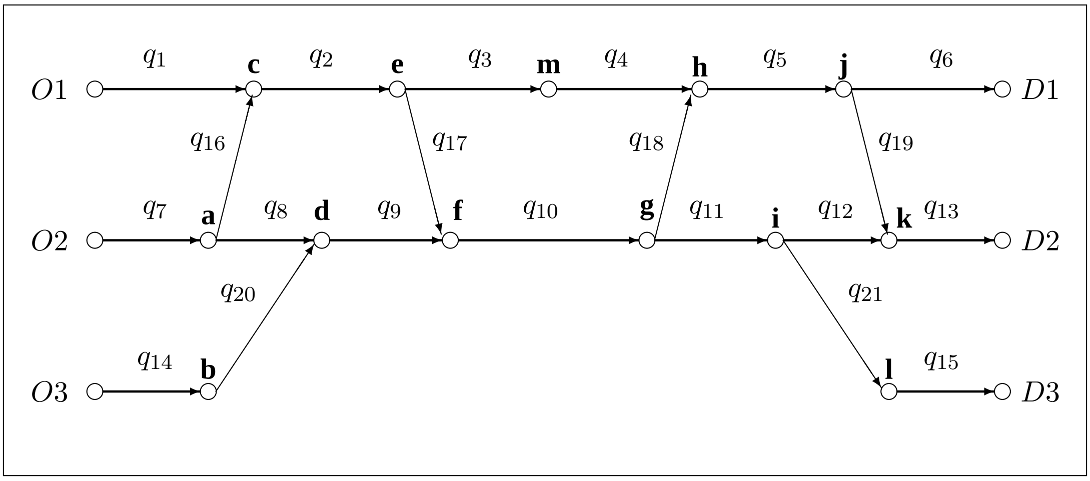


Figure 7.6: DWTC-CP/40 Project Network

	D1	D2	D3
O1	70%	15%	15%
O2	15%	70%	15%
O3	50%	50%	-

Table 7.2: Origin-Destination matrix,m_{rs}.

arc	d_a	road type	arc	d_a	road type
#	km		#	km	
1	2.3	highway	11	1.2	national
2	1.1	highway	12	0.9	national
3	2.3	highway	13	1.8	national
4	2.3	highway	14	1.8	city
5	1.1	highway	15	1.8	city
6	2.3	highway	16	0.5	national-highway $\Rightarrow$ national
7	1.8	national	17	0.5	highway-national $\Rightarrow$ national
8	0.9	national	18	0.5	national-highway $\Rightarrow$ national
9	1.2	national	19	0.5	highway-national $\Rightarrow$ national
10	3.6	national	20	0.5	city-national $\Rightarrow$ city
			21	0.5	national-city $\Rightarrow$ city

Table 7.3: Lengths of road segments.

path	arc	origin	destination	path length
k	a	r	s	km
1	1-2-3-4-5-6	1	1	11.4
2	1-2-3-4-5-19-13	1	2	11.6
3	1-2-17-10-21-15	1	3	11.7
4	7-16-3-4-5-6	2	1	11.6
5	7-16-2-3-4-5-19-13	2	2	11.8
6	7-8-9-10-11-21-15	2	3	11.5
7	14-20-9-10-18-5-6	3	1	11.7
8	14-20-9-10-11-12-13	3	2	11.5
9	1-2-17-10-18-4-5-6	1	1	11.8
10	1-2-17-10-11-12-13	1	2	11.6
11	7-8-9-10-18-5-6	2	1	11.6
12	7-8-9-10-11-12-13	2	2	11.4
13	7-16-2-17-10-18-5-6	2	1	12.0
14	7-16-2-17-10-11-12-13	2	2	11.8
15	7-8-9-10-18-5-19-13	2	2	11.8
16	7-16-2-17-10-18-5-19-13	2	2	12.2
17	7-16-2-17-10-11-21-15	2	3	11.9
18	14-20-9-10-18-5-19-13	3	2	11.9

Table 7.4: Paths between origins and destinations.

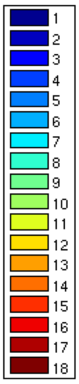


Figure 7.7: The colour code of the different paths

	Exit: arc number (%)															
Entrance name	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
Daussoulx	2	2	2	5	12	3	12	5	5	4	5	5	1	10	10	17
Eghezee	0	2	2	5	8	2	7	5	5	3	3	6	1	20	15	16
Thorembais	0	0	3	3	12	4	7	2	5	4	6	8	1	15	15	15
Walhain	0	0	0	4	10	2	8	1	4	2	3	2	1	15	18	30
Corroy	0	0	0	0	15	1	8	4	4	4	3	2	1	15	18	25
LLN	0	0	0	0	0	3	10	4	5	1	4	5	1	16	20	30
Louvranges	0	0	0	0	0	0	10	4	5	5	4	5	1	16	20	30
Wavre	0	0	0	0	0	0	0	4	6	4	7	7	1	20	24	27
Bierge	0	0	0	0	0	0	0	0	5	5	4	2	1	20	23	40
Rosiere	0	0	0	0	0	0	0	0	0	5	3	5	12	20	20	35
Overrijse	0	0	0	0	0	0	0	0	0	0	5	5	15	25	20	30
Argenteuil	0	0	0	0	0	0	0	0	0	0	0	0	20	20	30	30
Hoeilaart	0	0	0	0	0	0	0	0	0	0	0	0	20	20	30	30
Tervuren	0	0	0	0	0	0	0	0	0	0	0	0	0	0	40	60

Table 7.5: E411 OD matrix, m_{rs}.

7.3.3 E411 network

Highway E411 and national road N4 are represented by an oriented road network, see Figure 7.8. Here, we consider the portion between the Daussoulx junction and the junction of the Ring, (a_1 and a_{49} to a_{58})and a portion of the Ring (between Argenteuil entrance and the Wezembeek-Oppem exit). The network also includes a portion of the national road, N4 (a_{59} to a_{68}), between the Carrefour de Didi and Jezus-Eik. The cities of Hoeilaart and Tervuren (N3) are also present.

The network is described with the following characterisation. There are twenty-three entrances (a_1 to a_{23}) and fifteen exits (a_{24} to a_{30}) mainly connected on N4 and on the Ring. An highway arc has a capacity of 6000 veh/h (with maximum speed limit of 120 km/h), national roads and other city roads capacities are 2000 veh/h (various speeds limit from 50 km/h to 90 km/h). Some secondary roads capacities are 4000 veh/h. All entrances and exits have a length of 1000 m.

In most of the cases, paths choice pre-selection are limited to reasonable paths which allow to travellers to select between several different paths between their origin and their destination. Users between Eghezee and LLN can transit via the E411 or the N4 until Louvrange, but cannot switch from one to the other. After Louvrange, they can take either N4 or E411 until Jezus-Eik without possible alternative. Travellers from Argenteuil (RO), Tervuren or Hoeilaart do not have path choice.

Input values are based on MET measurements, (MET, 2003), only for E411, all other values are random values. The origin-destination matrix values are random percentages of distribution (see Tab.7.5). To simulate congestion, we chose from the MET measurements, data from peak hours. There are no available measurements for the RO. In that case, the flow rate is fixed at 90% of the road capacity (3600 veh/h).

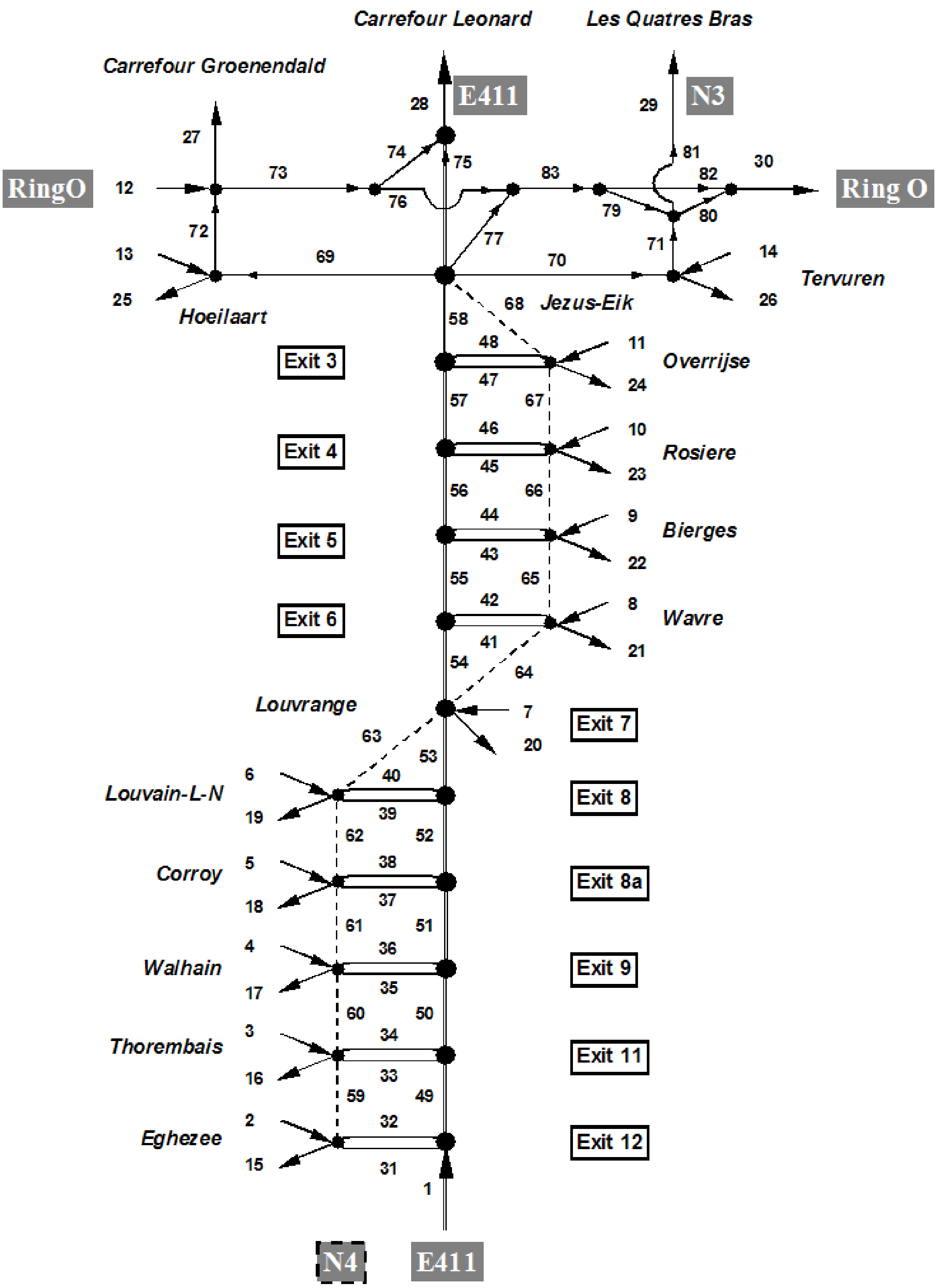


Figure 7.8: E411 network, arcs number.

7.4 The steady state maximum flow

Before to deal with the sustainable cost issue for a given demand, we first consider the problem of maximising the flow through a network. This aims to ensure that the demand does not exceed the maximum flow capacity of the network. The maximum flow problem is considered in steady-state conditions when the traffic is not congested. By "steady-state", we mean that the traffic variables $u(x,t), \rho(x,t)$ and $q(x,t)$ are constant with respect to the independent variables x and t over each arc of the network. Furthermore, no congestion means that the constant density and the constant flow are smaller than the critical values ($0 \leq \rho \leq \rho^*$, $0 \leq q \leq q_c$) while the speed is equal to its maximal value $u = u_{\text{max}}$. The maximum flow problem is solved using linear programming.

7.4.1 Formulation of the problem

Let us define the matrix M of all origin-destination pairs such that an element m_{rs} gives the proportion of the flow entering at origin r that exits at destination s and such that

$$\sum_{s \in D} m_{rs} = 1 \quad \forall r \in O. \tag{7.2}$$

Table 7.2 is an example of such an OD matrix. Let v_r denote the fraction of the total flow through the network originating from r and such that

$$\sum_{r \in O} v_r = 1. \tag{7.3}$$

The steady state flow on a path k is denoted h_k. The maximum flow problem is formulated as:

$$\max_{h_k} \quad \sum_{r \in O} \sum_{s \in D} \sum_{k \in K_{rs}} h_k \tag{7.4}$$

subject to

$$\text{arc flow capacity} \qquad \sum_{r \in O} \sum_{s \in D} \sum_{k \in K_{rs}} \delta_{ak} h_k \leq Q_a \qquad \forall a \in A \tag{7.5}$$

$$\text{path flow nonnegativity} \qquad h_k \geq 0 \qquad \forall k \in K \tag{7.6}$$

where Q_a is the maximal possible flow on arc a and

$$\delta_{ak} = \begin{cases} 1 & \text{if arc } a \text{ belongs to path } k \\ 0 & \text{otherwise.} \end{cases}$$

The left part of the first constraint (7.5) represents the relation between the path flow h_k and the arc flows q_a. The flow on an arc a, must be smaller than its capacity Q_a.

In order to satisfy the OD matrix, the objective function (7.4) may also be subjected to

$$\text{satisfaction of O-D matrix} \qquad \underbrace{\sum_{k \in K_{r,s}} h_k}_{\text{flow from } r \text{ to } s} = m_{rs} \underbrace{\sum_{x \in D} \sum_{k \in K_{rx}} h_k}_{\text{flow from } r} \qquad \forall r \in O, \forall s \in D \tag{7.7}$$

$$\begin{array}{l}\text{satisfaction of the flow} \\ \text{distribution among the origins}\end{array} \qquad \underbrace{\sum_{s \in D} \sum_{k \in K_{rs}} h_k}_{\text{flow from } r} = v_r \underbrace{\sum_{r \in O} \sum_{s \in D} \sum_{k \in K_{rs}} h_k}_{\text{total flow}} \qquad \forall r \in O. \tag{7.8}$$

The problem is solved with MATLAB using the linear programming function "linprog".

arc	case 1	case 2	case 3	case 4
#	speed (km/h)			
1	120	120	120	120
2	120	120	90	90
3	120	90	90	70
4	120	90	90	70
5	120	120	90	90
6	120	120	120	120

Table 7.6: Speed limitations on the highway.

7.4.2 Results

DWTC/40 project network

In this section, we present some results of the maximum flow problem for the DWTC/40 project network (see Fig. 7.6). We consider the problem with constraints (7.5), (7.6), (7.7) but *without* the constraint (7.8). With the OD matrix constraint (7.7), the distribution of flows among the arcs is represented in Fig.7.9 (case 1) and the maximum input is 9793 veh/h (see Tab. 7.7). Arcs 2,5 and 10 are saturated.

In order to evaluate the influence of speed limitations on the maximum flow, we consider three additional situations presented in Table 7.6. More precisely, we consider speed limitations on the highway namely:

- case 2: 90 km/h on arcs 3 and 4;
- case 3: 90 km/h on arcs 2,3,4,5;
- case 4: 90 km/h on arcs 2,5 and 70 km/h on arcs 3,4.

The results are given in Table 7.7 and illustrated in Fig. 7.9. We observe that the speed limitations induce only a small decrease of the maximum flow (from 9793 veh/h to 9299 veh/h) although the speed limitations are rather significant. In Fig. 7.10, the distribution of the flow among the various possible paths is represented. We observe that, for each OD pair, almost all the traffic takes the most natural and direct path. However, we remark that the flow from origin 2 to destination 2 (city-city) must be split between the highway and the national road in order to achieve the maximum flow.

Another interesting situation may be investigated. We consider the case where all the drivers use only the shortest path (in time) between an origin and a destination. In this case, the maximum flow becomes 6990 veh/h (case 1 in Table 7.7), which is significantly less than the optimal solution. One important observation is that the total flow may be higher if speed limitation is used (7713 veh/h for case 4). The reason can be seen in Fig. 7.11 and 7.12:with the speed limitations the paths passing by the highway need more time and some drivers have to change their path between the origin and the destination. Let us consider for instance the drivers going from origin 3 to destination 2:in the first case, they take the path 18 passing through arc 5 which is saturated; in case of speed limitation, the drivers use path 8 leaving some flow for arc 5 available for other drivers. We can see here how the speed limitations can be used as a tool for increasing the flow through a road traffic network.

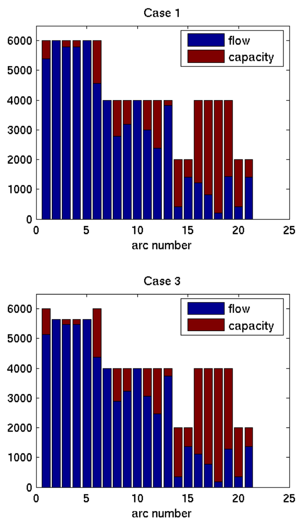


Figure 7.9: Max flow results for DWTC-Project network: the flows on the different arcs.

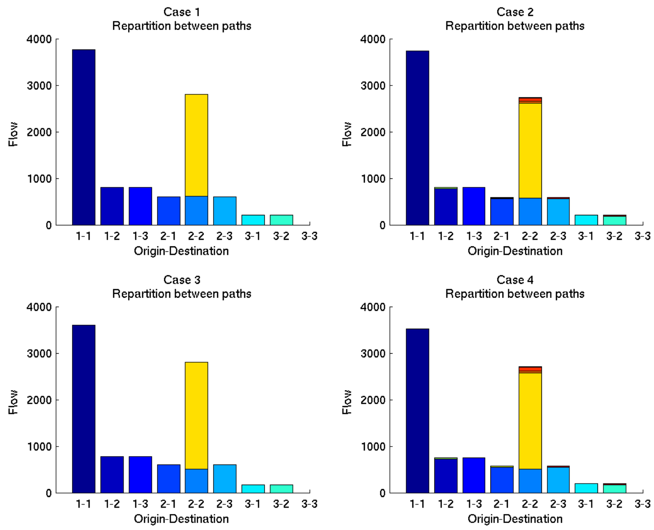


Figure 7.10: Max flow results for DWTC-Project network: distribution among the different paths.

Case	max flow (all paths)	max flow (shortest paths)
1	9793	6990
2	9651	6983
3	9480	7472
4	9299	7713

Table 7.7: The maximal flow for the DWTC/40 project network

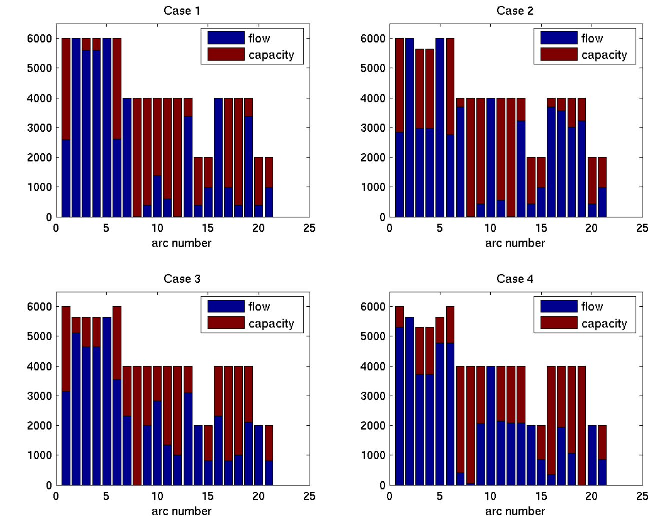


Figure 7.11: Max flow results for DWTC-Project network: the flow on the different arcs where the shortest path in time are used by all the drivers.

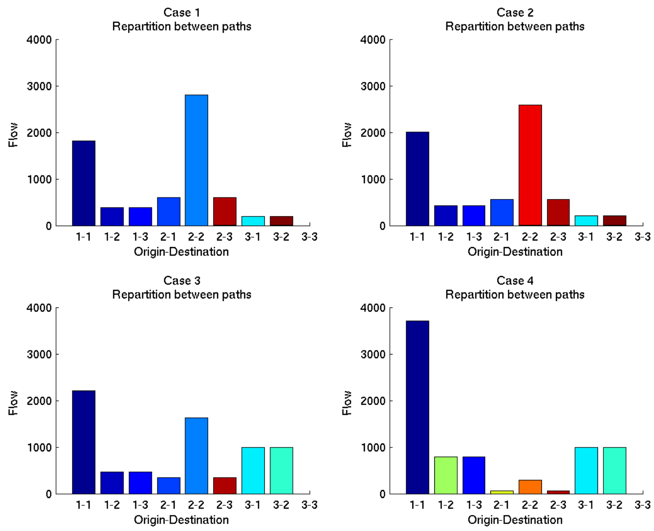


Figure 7.12: Max flow results for DWTC-Project network: the repartition between the different paths. The shortest path in time is used by the drivers.

E411 network

With the maximum speed limit on the network, the maximum flow with maximum speed is evaluated to 17 874 veh/h. The use of N4 increase near the Ring, and is almost saturated. The E411 access to the Ring (a_{77}) on Fig.7.8) is not saturated which imply that the vehicle distribution employ deviation roads (via Hoeilaart and Tervuren), see Figure 7.13.

If speed limitations are imposed to E411 near the Ring (see Table 7.8), the maximum flow is slightly decreasing (see Table 7.9).

case 1	case 2	case 3	case 4
speed (km/h)			
120	100	90	70

Table 7.8: Speed limit for the arcs 56, 57 and 58 on E411.

Case	max flow
1	17874
2	17813
3	17750
4	17581

Table 7.9: The maximal flows for the E411 network.

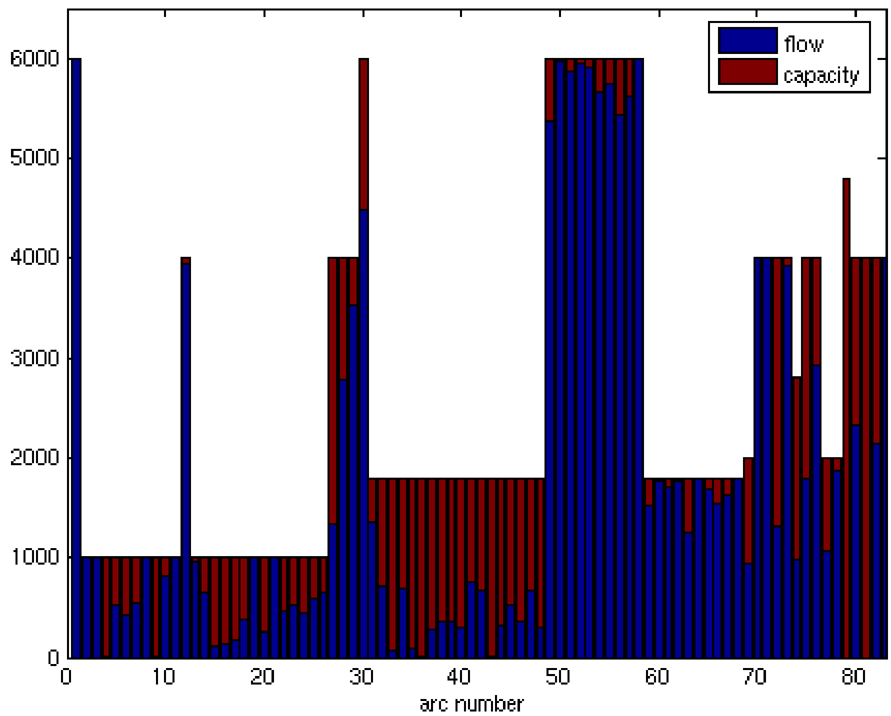


Figure 7.13: Vehicle flow and capacities on arcs for E411 network. Arcs # 49 to 58 are E411, Arcs # 59 to 68 are N4.

7.5 Sustainable cost function - steady state case

Sustainability can be defined as an economic state where the people and production demands agree with the preservation of the environment capacity for the future generations ([23]). In this project the issue of sustainability consists in finding the optimal use of the traffic network, in order to reduce the contributions to pollution and noise emission, while satisfying safety and effectiveness for the vehicles. There is an opposition between the user's point of view (reduction of travel time by increasing the speed) and the environment point of view because noise and pollution emissions increase with the speed. In order to analyse the trade-off between these points of view we define a sustainable cost function incorporating the econometric costs such as time cost, pollution and accident cost. Of course the relative costs, in monetary units, affected to each of these aspects reflect a political decision.

In section 7.5.1, 7.5.2, 7.5.3 we define the unit costs for one vehicle, respectively for time, pollution and accident costs, as well as the corresponding costs for the network per hour of operation. The evaluation of the unit costs is based on data provided by KUL/ETE. For the cost evaluation per hour of operation (expressed in € per hour), we consider in this section the steady state situation: on each arc the traffic conditions are uniform and do not depend on time. This implies that the speed on each arc is known, either equal to the corresponding u^{max} or equal to the imposed speed limit, and does not depend on the flow on the arc.

This sustainable cost function will be used in Section 7.6 in order to solve the optimisation problem consisting in determining the steady state traffic conditions minimising this sustainable cost function, while satisfying the constraints.

7.5.1 Time cost

Unit time cost

The "time cost" includes the direct vehicle operating cost (cost of fuel and additional running costs including repair and maintenance as a factor of fuel cost) as well as the time cost of vehicle occupants.

The definition of unit time cost in monetary value is based on Value of Time units (VOT, [17]), expressed in € per h and per vehicle.

The nominal VOT corresponds to the nominal travel time of the vehicle. If the travel time exceeds this nominal value (overtime), the value of the time cost increases up to VOT^*, for the part exceeding the nominal time.

The VOT depends also on the type of the journey (commuter, business, trucks, etc.). In this report we use an average unit cost computed, according to the proportions of the journey types on the Belgian roads. The unit cost for an equivalent vehicle reflecting these proportions, for nominal and overtime, are defined as weighted sums of the VOT:

$$C_{nom} = \sum_{\varrho \in R} \alpha_\varrho (VOT)_\varrho \text{ and } C_{ot} = \sum_{\varrho \in R} \alpha_\varrho (VOT)^*_\varrho \tag{7.9}$$

where R is the set of journey types. The values of α_ρ and (VOT), $(VOT)^*$ are given in Table 7.10, according to the data provided by ETE.

The resulting unit cost are given by

	commuting	business	leisure	HGV
α	0.48	0.03	0.44	0.06
VOT	6	21	4	40
VOT^*	9	31.5	4	40

Table 7.10: Time costs: proportion and VOT for different journey types, in € per hour per vehicle, Dunkerley & Prooft, 2004.

$$C_{nom} = 7.61 \left[\frac{€}{veh \cdot h}\right] \qquad C_{ot} = 10.15 \left[\frac{€}{veh \cdot h}\right].$$

Computation of the total time cost of a network

The travel time along a path k, between origin r and destination s (i.e. $k \in K_{rs}$) is given as

$$t_k = \sum_{a \in A} \delta_{ak} \frac{d_a}{u_a} \tag{7.10}$$

where d_a is the length of arc a, and u_a is the known speed on this arc.

In order to take into account the difference of VOT between the nominal and the overtime parts, it is necessary to decompose the travel time into these two components. The nominal travel time along k (no speed limitation) is

$$t_k^{nom} = \sum_{a \in A} \delta_{ak} \frac{d_a}{u_a^{max}}.$$

The overtime due to speed limitation on arc a being

$$\Delta t_a = \frac{d_a}{u_a} \left(1 - \frac{u_a}{u_a^{max}}\right),$$

it implies that the total overtime due to speed limitation on path k is

$$t_k - t_k^{nom} = \sum_{a \in A} \delta_{ak} \frac{d_a}{u_a} \left(1 - \frac{u_a}{u_a^{max}}\right) = \sum \delta_{ak} \Delta t_a.$$

This is illustrated in Figure 7.14.

Defining the nominal time from r to s as

$$t_{rs}^{nom} = \min_{k \in K_{rs}} t_k^{nom},$$

the overtime due to the choice of path k is

$$\Delta t_k^{nom} = t_k^{nom} - t_{rs}^{nom} \qquad k \in K_{rs}.$$

With these definitions, we have that the travel time on path k can be expressed as

$$t_k = t_{rs}^{nom} + \Delta t_k^{nom} + \sum_a \delta_{ak} \Delta t_a$$

and the cost of the travel along path k is given by

$$C_k = C_{nom} t_{rs}^{nom} + C_{ot}[\Delta_k^{nom} + \sum_a \delta_{ak} \Delta t_a] \left[\frac{€}{\text{veh}}\right].$$

The global time cost for one hour of network operation (€/h) is thus

$$\begin{aligned} J_T &= \sum_{k \in K} h_k C_k \\ &= \sum_{k \in K} h_k \left\{ C_{nom} t_{rs}^{nom} + C_{ot} \left[\Delta t_k^{nom} + \sum_{a \in A} \delta_{ak} \Delta t_a^{nom} \right] \right\} \left[\frac{€}{\text{h}}\right]. \end{aligned}$$

It can be seen, that under our assumptions, this cost is linear in the variables h_k.

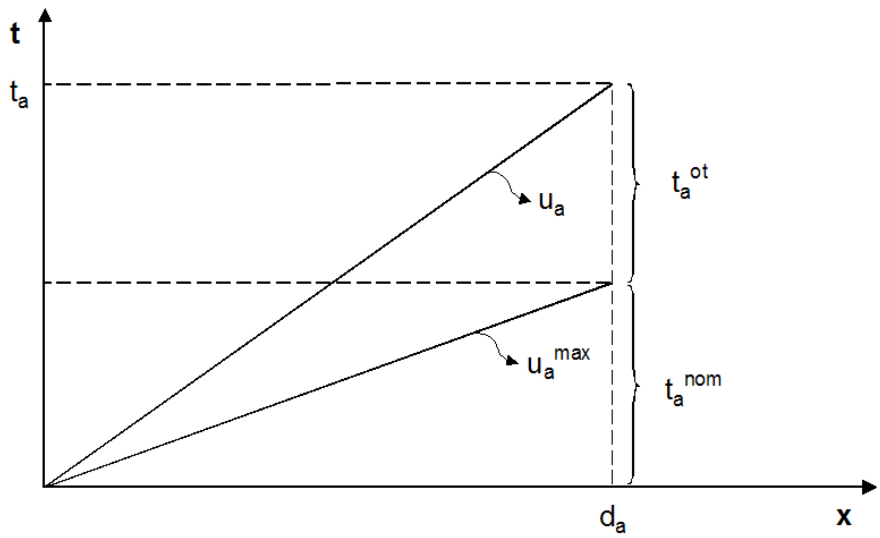


Figure 7.14: Overtime due to speed limit ($u_a < u_a^{max}$).

7.5.2 Pollution costs

Definition of pollution costs

Pollution cost function expresses in monetary value the social cost (diseases and deaths, lost days of work, etc.) which is associated to the pollution gases emission. The function is based on various parameters such as car type, pollutant type, car age. Emission factors are evaluated for various pollutants (CO_2, CO, NO_x, benzene, hydrocarbon, butadiene 1,3 and dust), by vehicles type. Vehicle categories are petrol and diesel cars, LGV, HGV[1], buses and their proportion according to the Belgian fleet. Ages of these vehicles is also taking into account with the EU standards (pre-Euro to Euro IV). Finally, each pollutant has a cost associated to a roadway type (highway, secondary road and city roads).

For each pollutant and each type of vehicle, there is an emission factor giving the mass of pollutant emitted for 1 km travel by this type of vehicle. This factor is a function of the speed. Figure 7.15 provides the typical examples of emission factors (expressed in g/km), in function of speed, for various

[1]large truck used for transporting goods.

gases and vehicles. Most of the pollutant present high values under 30 km/h and over 90 km/h. Thus low speed associated to congestion effect contributes significantly to pollutant emission. Trucks and buses exhibit the highest pollution emission. PM emission represents the dust emission.

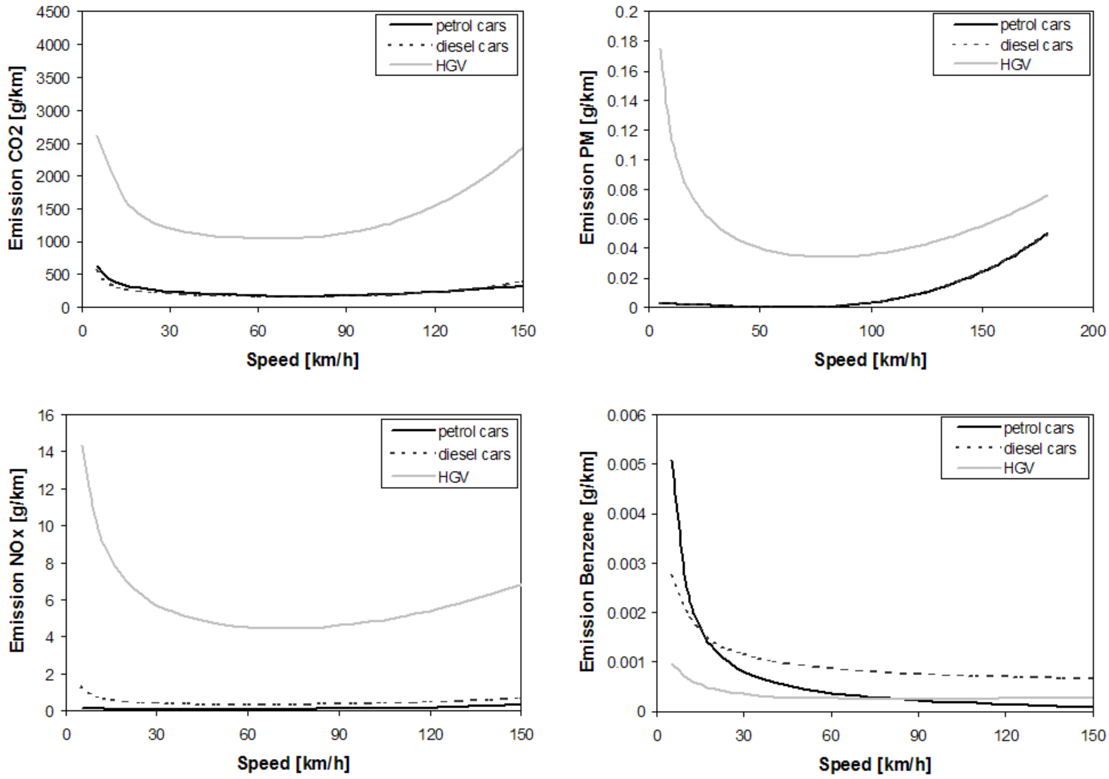


Figure 7.15: Pollutant emission factors, value (g/km) in function of speed for different vehicles types (cars with 2.0 l, Euro IV, except for CO_2 Euro II).

For the free flow conditions, when the speed is known independently of the traffic conditions, the pollution cost per km on a given arc depends therefore on the various emission factors, on the proportion of vehicles types and on the economical cost associated to each pollution according to the type of road (highway, secondary or city roads). It is expressed as

$$C_a^P = \sum_{\sigma \in S} \alpha_\sigma \left[\sum_{p \in \Pi} C_p(e_a) F_{p\sigma}(u_a) \right] \qquad \left[\frac{€}{\text{veh km}} \right] \tag{7.11}$$

In this expression

- Π is the set of pollutant types ;
- S is the set of vehicles types ;
- α_σ is the proportion of vehicles of type σ ;
- $F_{p\sigma}(u_a)$ is the emission factor for pollutant p and vehicle σ in g per vehicle and per km (see Fig. 7.15) ;

- e_a is the road type of arc a ;
- $C_p(e_a)$ is the cost of pollutant p (in €/g) for an arc of type e_a.

The values of the variables involved in this expression are deduced from data provided by KUL/ETE:

- the functions $F_{p\sigma}(u_a)$ see Fig. 7.15 ;
- the proportions of vehicles α_σ: 35% petrol cars, 35% diesel cars, 30% HGV ;
- the costs associated to each pollutant and each road type $C_p(e_a)$.

The resulting values of C_a^P, in function of the speed, for each road type are given in Fig. 7.16. The particular values corresponding to the speeds considered under steady state traffic are summarised in Table 7.11.

	120 [km/h]	90 [km/h]	70 [km/h]
highway	$2.3\,10^{-2}$	$1.55\,10^{-2}$	$1.42\,10^{-2}$
secondary		$0.83\;\;10^{-2}$	$0.72\,10^{-2}$
city			$1.82\;\;10^{-2}$

Table 7.11: Values (in $\frac{€}{veh \cdot km}$) of C_a^P for different road types and for different velocities.

Computation of the total pollution cost for a network

For a given arc a of length d_a, the pollution cost associated to an equivalent vehicle is equal to $C_a^P d_a$. For one hour of operation, corresponding to a flow of q_a vehicles on the arc, and for the whole network, the global pollution cost is given by

$$J_P = \sum_{a \in A} C_a^P \; q_a \; d_a \qquad [€/\text{h}] \tag{7.12}$$

It must be pointed out that, under an assumption concerning the speed on each arc, this cost is linear with respect to the flows q_A on the different arcs.

7.5.3 Accident costs

Definition of accident costs

Accident costs reflect the cost of collision between vehicles according to an accident risk. Costs are related to the willingness of users to pay for security and the part taken by the society.

The risk is defined in function of the number of accident between vehicles categories and the annual travelled distance-vehicle. Risk calculation depends also on vehicle speed u_a and on the type of accident $severity$ (fatal, severe, and light). Vehicle types represent are car, bus and HGV.

Based on this statistical risk evaluation, an accident cost constant C^A can be evaluated depending on the speed u_a on arc a; according to the data provided by KUL/ETE

$$C^A(u_a) = 1.665\,10^{-3} u_a^2 + 1.7\,10^{-5} u_a^3 \quad [€/\text{veh km}].$$

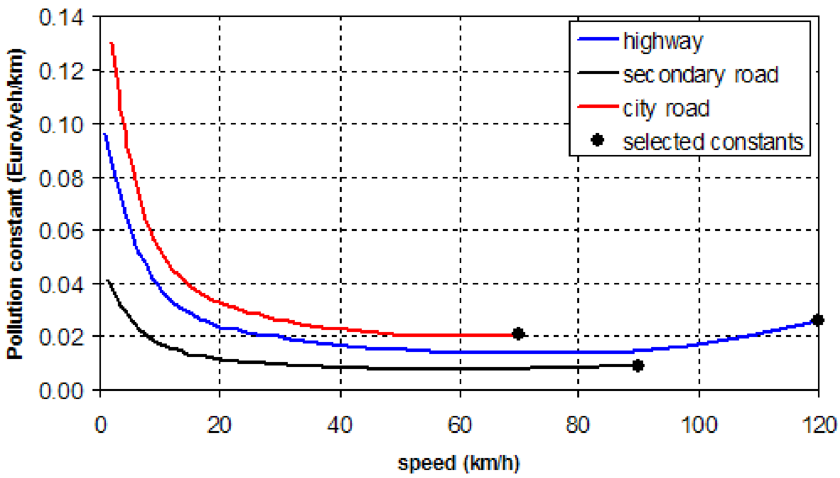


Figure 7.16: Pollution constants value in function of speed for different road types.

Computation of the global accident cost for a network

As for the pollution cost, the accident cost on the network is computed as

$$J_A = \sum_{a \in A} C^A(u_a) q_a d_a. \tag{7.13}$$

7.5.4 Global cost evaluation

The global cost function includes the 3 components presented in sections (7.5.1), (7.5.2) and (7.5.3):

$$J_G = J_T + J_P + J_A \tag{7.14}$$

This global cost is a linear combination of the variables h_k and q_a. The flow on arcs q_a can also be expressed as a linear combination of the path flow h_k. The global cost is thus linear with respect to h_k. This expression will be used in the next section as the objective function for the optimization of the flow distribution on the network.

7.6 Minimum Sustainable Cost - Steady state

In this section we address the problem of the optimal distribution of the traffic flows among all possible paths, for a given demand. We consider the steady-state situation and the optimality criterion is the minimisation of the global sustainability cost function J_G defined in section 7.5. It can be expected that the minimal sustainable cost should be achieved under traffic conditions without jams. This implies that the speed on each arc is equal either to u_a^{max}, (the normal speed corresponding to the horizontal portion of the speed density relation), or to an imposed u_a, in case of speed limitation. The set of speeds on the different arcs can therefore be considered as given constant parameters of the problem.

The minimisation problem is described as follows:

$$\min_{h_k} J_G = \min_{q_a, h_k} \left\{ J_T + J_P + J_A \right\} \tag{7.15}$$

subject to

a given demand	$\sum_{r\in O}\sum_{s\in D}\sum_{k\in K_{rs}} h_k = F_{in}$	
satisfaction of O-D matrix	$\sum_{k\in K_{r,s}} h_k = m_{rs}\sum_{x\in D}\sum_{k\in K_{rx}} h_k$	$\forall r \in O, \forall s \in D$
satisfaction of the flow distribution trough origins	$\sum_{s\in D}\sum_{k\in K_{rs}} h_k = v_r\sum_{r\in O}\sum_{s\in D}\sum_{k\in K_{rs}} h_k$	$\forall r \in O.$

As in Section 7.4, it is a linear programming problem which is solved with MATLAB using the linear programming function "linprog".

7.6.1 Tests on networks

DWTC/40 project network

We summarise hereafter optimisation results corresponding to various conditions:

1. We consider, as in Section 7.4, four cases corresponding to different speed limitations (see Table 7.6).

2. The total demand is fixed at 7000 veh/h (4000 from origin O_1, 2000 from O_2 and 1000 from origin 0_3), with the m_{rs} matrix considered in Section 7.4 (see Table 7.2).

   It must be pointed out that, for case 1 and case 2, the global flow exceeds the maximum flow achievable if each user follows its shortest path (see Table 7.7). Several paths between each O-D pair are therefore necessary in order to satisfy the demand. On the other hand, this demand is less than the maximum possible flow, this implies that the network is not saturated and several degrees of freedom are available in order to minimise the cost criterion.

3. The optimisations are performed using 3 types of criterion: minimisation with respect to the global cost J_G, with respect to the time cost J_T or to the pollution cost J_P.

The results are summarised in Table 7.12, giving for each minimisation criterion and each case the values of the different components to the cost (time, pollution and accident) and the value of the global cost. It can be observed that the time cost component is the most determinant and that the accident cost is very low. As expected the effect of speed limitations is to increase the time cost and to decrease the pollution and accident costs.

Minimisation with respect to the global cost or to the time cost only produce almost the same values for the cost value, and mostly the same repartitions between the possible paths. The reason is that the time cost represents the major part of the global cost. If we minimise with respect to the pollution cost, the pollution cost decreases, at the price of an augmentation of the time cost and of the global cost.

It is possible to represent the flows on the different arcs and on the different paths. Fig.7.17 and 7.18 give these results for the 4 cases, when the optimisation is performed with respect to the global cost. In case 1, saturation occurs on the highway (arc 5) while for the 3 other cases saturation occurs on arc 10. The effect of the speed limitations on the repartition between the possible paths for all O-D pairs is represented on Fig.7.18. Consider for instance the (2-2) pair. In case 1 all users follow path 5, for case 2 most follow path 14, while in case 2 and 4 most use path 12.

Fig.7.19 represents the flows on the arc, when the optimisation is performed with respect to the pollution cost. The flow repartition can be compared with Fig. 7.17. In this situation, saturation occurs on the same arc (arc 5) for the 4 cases. In fact the flow on the highway decreases with augmentation of the middle branch flow. In order to minimise pollution we privilege the arcs with smaller velocity and more drivers avoid the highway, accepting travel time augmentation.

criterion	case	Global Cost	Time Cost		Pollution Cost		Accident Cost	
		[€/h]	[€/h]	%	[€/h]	%	[€/h]	%
Global cost	1	6793	5171	76	1491	22	131	2
	2	7603	6246	82	1236	16	121	2
	3	8003	6778	85	1111	14	115	1
	4	8870	7667	86	1093	12	110	1
Time cost	1	6793	5171	76	1491	22	131	2
	2	7603	6246	82	1236	16	121	2
	3	8003	6778	85	1111	14	115	1
	4	8870	7667	86	1093	12	110	1
Pollution cost	1	6995	5583	80	1281	18	131	2
	2	7657	6361	83	1175	15	121	2
	3	8003	6778	85	1111	14	115	1
	4	8870	7667	86	1093	12	110	1

Table 7.12: The different costs for the DWTC/40 project network.

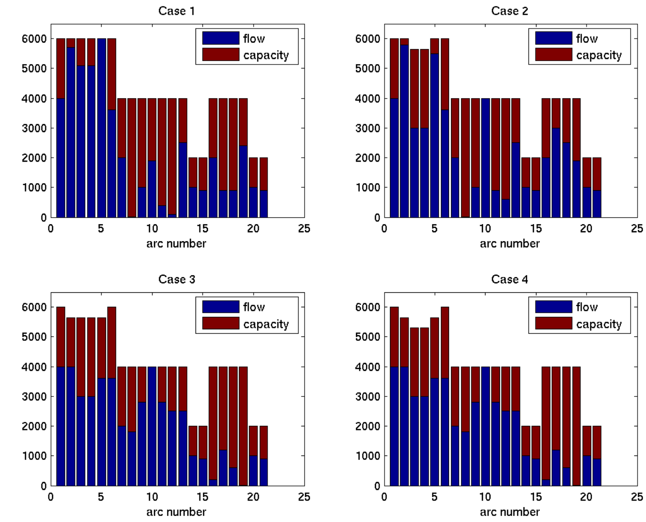


Figure 7.17: Vehicle flow on arcs for DWTC-CP/40, optimization with respect to the global cost.

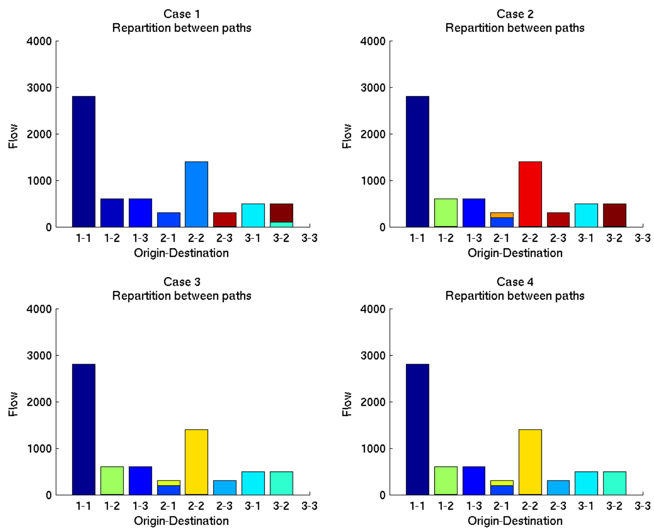


Figure 7.18: Repartition of the flows between the possible paths for DWTC-CP/40, optimization with respect to the global cost.

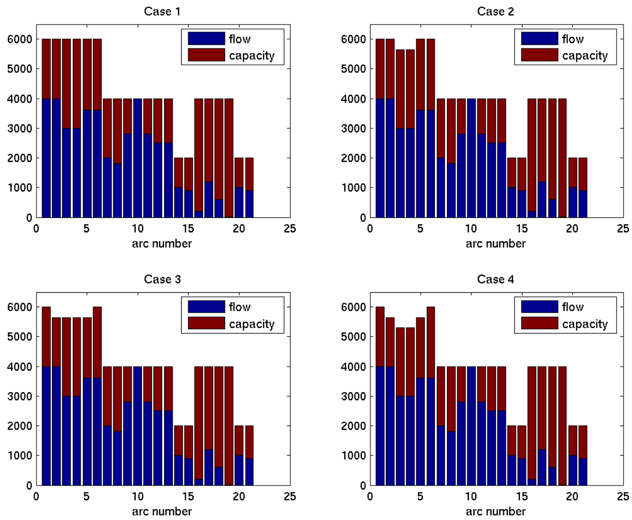


Figure 7.19: Vehicle flow on arcs for DWTC-CP/40, optimization with respect to the pollution cost.

E411 network

The sustainable costs and the effects of speed limitations can be computed for the E411 network. If we choose a fixed distribution of the flow through the network and compute the different costs for the four speed limitations described in Table 7.8, we obtain the results of Table 7.13. The fixed distribution chosen is the one maximising the flow for case 4 (17581 veh/h).

case	Global Cost	Time Cost		Pollution Cost		Accident Cost	
	[€/h]	[€/h]	%	[€/h]	%	[€/h]	%
1	30699	22943	75	7029	23	728	2
2	31347	23864	76	6780	22	703	2
3	31831	24478	77	6660	21	692	2
4	33505	26233	78	6598	20	674	2

Table 7.13: The different costs for the E411 network.

As expected, we can also notice here the trade-off between the time cost and the pollution cost. With speed limitation, the pollution cost decrease but the time cost increase.

7.7 Sustainable cost function evaluation for transient traffic

In this section, our purpose is to evaluate the evolution of the global sustainability cost function, for a transient traffic situation such as simulated by the simulation tool described in section 7.8. The steady-state free-flow traffic condition, assumed in section 7.5 and 7.6, are relaxed and this implies that all the traffic variables (speed, density, flow) are time dependant and not uniform along a given arc. The cost parameters C_a^P and C^A involved in the evaluation of pollution and accident costs, depending in fact on the speed, can not therefore be considered as constants and depend on the traffic conditions. Moreover the travel time along a path is no longer a constant and depends also on the traffic conditions. This justifies another approach to evaluate the 3 components of the sustainable cost function. These three components are evaluated arc by arc, independently of the particular paths.

7.7.1 Time cost

On a given arc a consider an arc portion dx, around the abscissa x $(0 \leq x \leq d_a)$ and a time interval dt, around time t. There are $\rho_a(x,t)dx$ vehicles on this arc portion.

During the time period dt each of this vehicles covers a distance equal to $u_a(x,t)dt$. On the other hand, under normal speed condition (i.e. $u_a(x,t) = u_a^{max}$), the time necessary to cover this distance would be equal to $\frac{u_a(x,t)}{u_a^{max}}dt$. We therefore decompose the time period dt in two parts:

$\dfrac{u_a(x,t)}{u_a^{max}}dt$	considered as the nominal part ;
$(1 - \dfrac{u_a(x,t)}{u_a^{max}})dt$	considered as an overtime due to the fact that $u_a(x,t) \leq u_a^{max}$.

The time cost for the vehicles on this arc portion, during this time period dt, is therefore equal to

$$C_{nom}\rho_a(x,t)\left[\frac{u_a(x,t)}{u_a^{max}}\right]dxdt + C_{ot}\rho_a(x,t)\left[1 - \frac{u_a(x,t)}{u_a^{max}}\right]dxdt = C_a(\rho_a(x,t), u_a(x,t))dxdt.$$

For a time period T_0 and for the whole network, the global time cost function is equal to

$$J_T[0,T] = \int_0^{T_0} \left[\sum_{a \in A} \int_0^{d_a} C_a(\rho_a(x,t), u_a(x,t)) dx \right] dt.$$

Equivalently, we can define a time cost rate.

$$\frac{dJ_T}{dt} = \sum_{a \in A} \int_0^{d_a} C_a(\rho_a(x,t), u_a(x,t)) dx \tag{7.16}$$

At each step of simulation process, this cost rate is evaluated by integration of $C_a(x,t)$ along all the arcs a. The global time cost is then the time integral of this cost rate and can be evaluated in parallel with the time integration of the traffic evolution.

Remark:

This expression can be used to evaluate the time cost, for one hour of operation and for steady-state free flow traffic, as in section 7.5. It is easy to check that it produces the same expression, except the fact that we loose the overtime associated to the choice of the path.

7.7.2 Pollution cost

Considering the arc interval dx, during the time period dt. The pollution cost of one vehicle is equal to $C_a^P(u_a)u_a(x,t)dt$, and for all the vehicles on the interval $C_a^P(u_a)u_a\rho_a dtdx$.

The total pollution cost, for the whole network during a time period T is given by

$$J_P[0,T] = \int_0^T \sum_{a \in A} \int_0^{d_a} \left[C_a^P(u_a(x,t))u_a(x,t)\rho_a(x,t) \right] dxdt.$$

Remark: Here again we can define the pollution cost rate as

$$\frac{J_P}{dt} = \sum_{a \in A} \int_0^{d_a} C_a^P(u_a)u_a\rho_a dx$$

where the unit cost $C_a^P(u_a)$ represents the pollution cost per covered kilometr, and therefore leads to infinity for u_a tending to zero. The product $(C_a^P(u_a)u_a)$ is the pollution cost per unit time (hour). This quantity tends to a finite value, for u_a tending to zero.

7.7.3 Accident cost

Similarly to the pollution cost, we define the accident cost function and the accident cost rate as

$$J_A[0,T] = \int_0^T \sum_{a \in A} \int_0^{d_a} \left[C^A(u_a(x,t))u_a(x,t)\rho_a(x,t) \right] dxdt$$

and

$$\frac{J_A}{dt} = \sum_{a \in A} \int_0^{d_a} \left[C^A(u_a(x,t))u_a(x,t)\rho_a(x,t) \right] dx.$$

7.7.4 Global cost

The global sustainable cost can therefore be evaluated in parallel with the numerical time integration provided by the simulator:

$$J_G[0,T] = J_T[0,T] + J_P[0,T] + J_A[0,T]$$

$$\frac{dJ_G}{dt} = \frac{dJ_T}{dt} + \frac{dJ_P}{dt} + \frac{dJ_A}{dt}$$

7.8 Dynamical Simulator

7.8.1 The LWR Road Network Model

The LWR single road model

The model used in this simulator is the LWR model [33]. This model is based upon the macroscopic variables introduced in Section 7.2. These variables are the density and the speed of the vehicles at position x and time t: $\rho(x,t)$ and $u(x,t)$. The dynamics of the traffic is represented by a conservation law expressed as

$$\frac{\partial \rho}{\partial t} + \frac{\partial(\rho u)}{\partial x} = 0. \tag{7.17}$$

The main assumption of the LWR model is that the drivers instantaneously adapt their speed in function of the surrounding density i.e.:

$$u(x,t) = U(\rho(x,t)). \tag{7.18}$$

The function $Q(\rho) = \rho U(\rho)$ is the “flow rate” representing the number of vehicles per time unit passing through a particular position in function of the traffic state at this position. Inserting (7.18) in (7.17), the LWR model is:

$$\frac{\partial \rho}{\partial t} + \frac{\partial Q(\rho)}{\partial x} = 0. \tag{7.19}$$

With this model, we don’t make any distinction between the different vehicles present at position x at time t. However it may be useful to make a distinction between different *classes* of vehicles. For example the vehicles with the same origin–destination path belong to the same class. Being able to distinguish the vehicles in function of their path will be very useful in order to describe the behaviour of the traffic at the junctions between different roads. If we have n different classes the LWR model (7.19) becomes

$$\frac{\partial}{\partial t}\begin{pmatrix}\rho^1\\ \vdots \\ \rho^n\end{pmatrix} + \frac{\partial}{\partial x}\begin{pmatrix}\frac{\rho^1}{\rho^{\text{tot}}}Q(\rho^{\text{tot}})\\ \vdots \\ \frac{\rho^n}{\rho^{\text{tot}}}Q(\rho^{\text{tot}})\end{pmatrix} = 0 \tag{7.20}$$

where $\rho^i(x,t)$ is the density of vehicle of class i in position x at time t and $\rho^{\text{tot}} = \sum_i \rho^i$ the total density.

A traditional problem studied for a conservation law of the form (7.19) is the Riemann problem which is an initial value problem where the initial condition consists of two constant values:

$$\rho(x,0) = \begin{cases}\rho_l & \text{if} \quad x \leq 0\\ \rho_r & \text{if} \quad x > 0.\end{cases}$$

The solutions of the Riemann problem may produce two types of waves (see Fig. 7.20):

- **shock wave** if $\rho_r > \rho_l$. This shock wave (a discontinuity in ρ) is moving at the speed $\frac{Q(\rho_r)-Q(\rho_l)}{\rho_r-\rho_l}$;
- **rarefaction wave** if $\rho_r < \rho_l$. The spatial interval occupied by the rarefaction wave (a continuous transition from ρ_l to ρ_r) at time t is $[Q'(\rho_l)t, Q'(\rho_r)t]$.

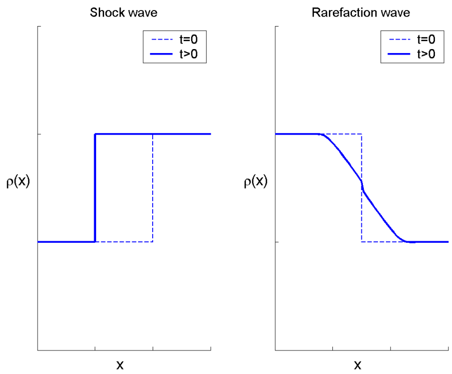


Figure 7.20: The waves present in the solution of a Riemann problem.

The Riemann problem is important, not only because it allows an explicit solution but also because the solution of any initial value problem with arbitrary initial conditions can be constructed from a set of appropriate Riemann problems (see e.g. [9]).

A numerical scheme for the LWR model

In order to solve the partial differential equation (7.19), we need to use an appropriate numerical scheme. Different schemes exist but the most simple and intuitive is the Daganzo scheme (see [14]).

In this scheme, the highway is divided into cells of equal size Δx. The scalar $\rho_i(k)$ represents the average density in cell i at time $k \cdot \Delta t$ where Δt is the discrete time step. The function $Q(\rho)$ is assumed to have only one maximum Q_{max} at the critical density ρ^*. We introduce two new functions:

$$S(\rho) = \begin{cases} Q(\rho) & \text{if } \rho \leq \rho^* \\ Q_{\text{max}} & \text{if } \rho > \rho^* \end{cases}$$

$$R(\rho) = \begin{cases} Q_{\text{max}} & \text{if } \rho \leq \rho^* \\ Q(\rho) & \text{if } \rho > \rho^* \end{cases}$$

where $S(\rho)$ is the *sending* function which represents the flow that the cell is able to send to the downstream cell and $R(\rho)$ the *receiving* function which represents the flow that the cell is able to receive from the upstream cell. These two functions are represented in Figure 7.21 for the triangular $Q(\rho)$ function.

With the descriptions of the functions $S(\rho)$ and $R(\rho)$, it is obvious that the flow $F_{i+\frac{1}{2}}$ between the cell i and the cell $i+1$ is

$$F_{i+\frac{1}{2}} = \min(S(\rho_i), R(\rho_{i+1})). \tag{7.21}$$

The densities in the cells are updated according to the rule:

$$\rho_i(k+1) = \rho_i(k) + \frac{\Delta t}{\Delta x}\left(F_{i-1/2} - F_{i+1/2}\right).$$

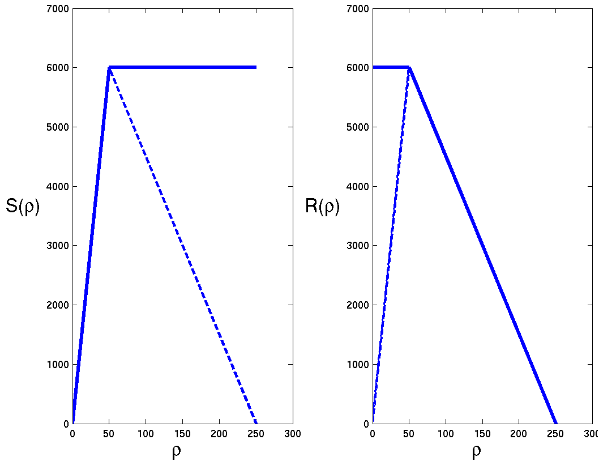


Figure 7.21: The sending and receiving function for the triangular $Q(\rho)$ relation.

It can also be shown that (7.21) is the exact value of the flow at $x = 0, t > 0$ for the following Riemann problem

$$\rho(x,0) = \begin{cases} \rho_i & \text{if} \quad x \leq 0 \\ \rho_{i+1} & \text{if} \quad x > 0. \end{cases}$$

This is the reason why the Daganzo scheme converges to the exact solution of (7.19).

If now we consider the LWR model with different classes, we can easily extend the Daganzo scheme in orderto solve (7.20). First, we compute the flows between the cells by (7.21) where the sending and receiving capacities are evaluated on the basis of the total density $\rho^{\text{tot}} = \sum_j \rho^j$. Then, the density ρ_i^j is updated as follows:

$$\rho_i^j(k+1) = \rho_i^j(k) + \frac{\Delta t}{\Delta x}\left(F_{i-1/2}\frac{\rho_{i-1}^j(k)}{\sum_j \rho_{i-1}^j(k)} - F_{i+1/2}\frac{\rho_i^j(k)}{\sum_j \rho_i^j(k)}\right).$$

This extended Daganzo scheme is easy to implement but doesn't converge to the exact solution of (7.20) even when $\Delta x \rightarrow 0$. One may consider the particular initial conditions represented in Figure 7.22 where there are only two classes of vehicles. On the first part, there are only vehicles of class 2, while on the second, there is only vehicles of class 1. The initial density being constant everywhere, all the vehicles travel at the same speed, none can bypass another and the exact solution must just be the displacement of the discontinuity. The solution from the extended Daganzo scheme is represented in Figure 7.23. We can observe the presence of some numerical diffusion in this extended Daganzo scheme. The presence of this diffusion is not a big problem for two reasons:

1. The evolution of the total density is correctly evaluated.
2. In the reality, all the drivers don't have the same behaviour. We can thus argue that the presence of some numerical diffusion adds a little realism.

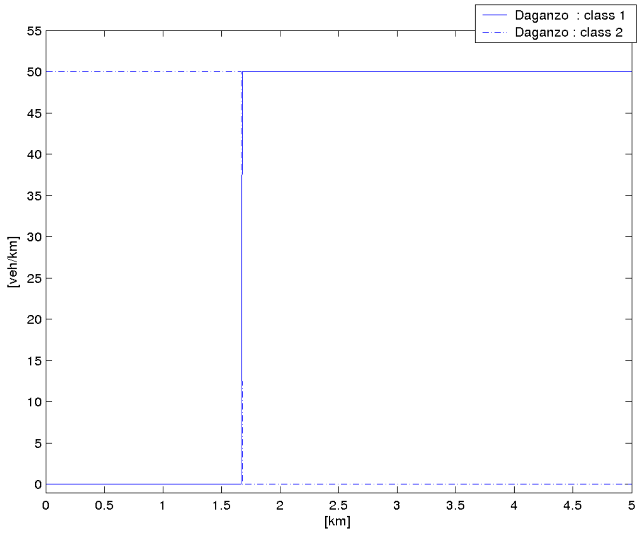


Figure 7.22: A special case where the initial total density is the same everywhere.

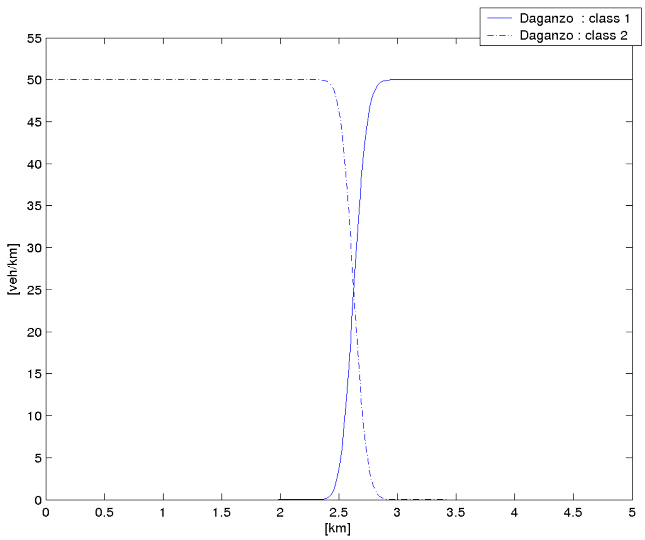


Figure 7.23: Illustration of the diffusion in the extended Daganzo scheme.

The junction models

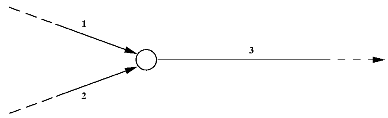
As explained in Section 7.3, a road network can be represented as a graph where each arc represents a road and each node a junction between roads. The different types of junction implemented in the simulator are represented in Table 7.14.

Naturally we assume that the density on each arc satisfies the single road LWR model. In order to complete the model, we need to describe the mechanism that occurs at the junctions. The modelling of these mechanisms has a significant influence on the global behaviour of the system.

nbr. of incoming arcs	nbr. of outgoing arcs	junction type
0	1	origin
1	0	end
1	2	diverge
2	1	merge
1	1	on-ramp
1	1	off-ramp
1	1	change of road properties

Table 7.14: The types of junction implemented in the simulator.

The merge junction



For the merge junction, we will present a complete description of the modelling in order to introduce all the concepts: sending and receiving capacity, priority factors,... The description of the other types of junction will be more concise.

Like for the single road model, one of the elementary problems we can study, and from which a global solution will be constructed, is the Riemann problem. For a Riemann problem at a junction, we take a constant density on the three roads as initial condition:

$$\rho_i(x_i, 0) = \rho_{i,0} \qquad i = 1, 2, 3. \tag{7.22}$$

A first restriction on the values of the densities on the borders of the junction is the conservation of flows:

$$\sum_{i=1}^{2} Q(\rho_i(0,t)) = Q(\rho_3(0,t)) \qquad \forall t. \tag{7.23}$$

However as it can be expected, for the system of conservation laws (7.19), the boundary condition (7.23) and the initial condition (7.22) are not sufficient to have a unique solution to the Riemann problem at the junction. In addition, some of the admissible solutions may be totally unrealistic. For instance, $\bar{\rho}_1 = \bar{\rho}_2 = \rho_{\max}$, $\bar{\rho}_3 = 0$ is always a possible mathematical solution although is is clearly counterintuitive. A natural way to have a unique solution is to add a supplementary condition.

Whatever is the supplementary condition, a restriction on the possible values of the densities at the junction will be present for all the models. The following reasoning explains the presence of this restriction:
In order to describe a solution of a Riemann problem at a junction, given the initial condition (7.22) at time t, the model must express the values of the new densities at the borders of the junction at time t^+:

$$\begin{aligned} \rho_i(0, t^+) &= \bar{\rho}_i \qquad i = 1, 2 \\ \rho_3(0, t^+) &= \bar{\rho}_3. \end{aligned}$$

After having specified the values of the densities at the borders of the roads, the solution of the Riemann problem at the junction will be based on the Riemann problems on the different roads:

$$\begin{cases} \rho_i(x_i,0) = \rho_{i,0} \quad \forall x_i < 0 \\ \rho_i(0,0) = \bar{\rho}_i \end{cases} \qquad i = 1,2$$
$$\begin{cases} \rho_3(0,0) = \bar{\rho}_3 \\ \rho_3(x_3,0) = \rho_{3,0} \quad \forall x_3 > 0. \end{cases}$$

Because the wave produced by the Riemann problem $(\rho_{i,0}$—$\bar{\rho}_i)$ on an incoming road must have a negative speed (to not immediately re-enter in the junction), the possible values for $\bar{\rho}_i$ must be restricted to a subset of $[0, \rho_{\max}]$. This is the same for the outgoing road where the speed of the waves produced by the Riemann problem $(\bar{\rho}_3$—$\rho_{3,0})$ must be positive. Based on the speed of the waves described in section 7.8.1, one can show that we must have

$$\bar{\rho}_i \in \begin{cases} \{\rho_{i,0}\} \cup]\tau(\rho_{i,0}), \rho_{\max}] & \text{if} \quad 0 \leq \rho_{i,0} < \rho^* \\ [\rho^*, \rho_{\max}] & \text{if } \rho^* \leq \rho_{i,0} \leq \rho_{\max} \end{cases} \qquad i = 1,2$$

$$\bar{\rho}_i \in \begin{cases} [0, \rho^*] & \text{if} \quad 0 \leq \rho_{i,0} \leq \rho^* \\ \{\rho_{i,0}\} \cup [0, \tau(\rho_{i,0})[& \text{if } \rho^* < \rho_{i,0} \leq \rho_{\max} \end{cases} \qquad i = 3$$

where for each $\rho \neq \rho^*$, $\tau(\rho)$ is the unique number $\tau(\rho) \neq \rho$ such that $Q(\rho) = Q(\tau(\rho))$. In terms of flow, the admissible regions for the flows are

$$Q(\bar{\rho}_i) \in [0, S_i] = \begin{cases} [0, Q(\rho_{i,0})] & \text{if } 0 \leq \rho_{i,0} \leq \rho^* \\ [0, Q(\rho^*)] & \text{if } \rho^* \leq \rho_{i,0} \leq \rho_{\max} \end{cases} \qquad i = 1,2$$

$$Q(\bar{\rho}_i) \in [0, R_i] = \begin{cases} [0, Q(\rho^*)] & \text{if } 0 \leq \rho_{i,0} \leq \rho^* \\ [0, Q(\rho_{i,0})] & \text{if } \rho^* \leq \rho_{i,0} \leq \rho_{\max} \end{cases} \qquad i = 3 \quad (7.24)$$

where S_i represents the “sending” capacity of an incoming road and R_i the “receiving” capacity of an outgoing road which were introduced in Section 7.8.1.

For the merge junction, the natural condition used in the simulator is the maximisation of the passing flow:

$$\sum_{i=1}^{2} Q(\bar{\rho}_i).$$

In the case where $S_1 + S_2 \leq R_3$, the maximum is unique

$$\begin{cases} Q(\bar{\rho}_i) = S_i & i = 1,2 \\ Q(\bar{\rho}_3) = S_1 + S_2. \end{cases}$$

If $S_1 + S_2 > R_3$, we must give some “priority factors” between the incoming flows. Several are possible (see [31]):

- the priority factors may be function of the incoming flows

$$\alpha_i = \frac{S_i}{\sum_{i=1}^{2} S_i};$$

- the priority factors may depend on some fixed coefficients p_i depending on the road geometry

$$\alpha_i = \begin{cases} p_i & R_3 p_i \leq S_i \\ \frac{S_i}{R_3} & S_i < R_3 p_i. \end{cases}$$

The possible passing flow is then split between the incoming roads in function of these priority factors:

$$\begin{cases} Q(\bar{\rho}_i) = \alpha_i R_3 & i = 1, 2 \\ Q(\bar{\rho}_3) = R_3. \end{cases}$$

The capacity drop phenomenon The *capacity drop phenomenon* is a critical phenomenon which represents the fact that the outflow of a traffic jam is significantly lower than the maximum achievable flow at the same location. We can easily understand this phenomenon in the case of a merge junction where two roads merge in one: if there are too much vehicles trying to access the same road, there is a sort of mutual embarrassment between the drivers which results in an outgoing flow lower than the optimal possible flow.

This phenomenon has been experimentally observed (see [11] and [21]). The flow decrease, which may range up to 15 %, has a considerable influence when considering traffic control ([44]). To have a model describing this phenomenon is thus a critical element in the establishment of a traffic state regulation strategy.

Unfortunately the criterion described in the previous paragraph which consists of maximising the passing flow:

$$\sum_{i=1}^{2} Q(\bar{\rho}_i)$$

under the constraints (7.24) does not represent the capacity drop phenomenon. Instead of maximising the criterion over the region defined by (7.24), we suggest to make the maximisation over a subregion. Defining

$$R_3' = \min\left(R_3, g\left(\sum_i S_i\right)\right)$$

where $g(x)$ is a function whose shape is represented in Figure 7.24. In this expression of R_3', we have

- $\sum_i S_i$ which represents the sum of the flows wishing to enter the outgoing road ;
- the function $g(\cdot)$ which expresses the fact that when too much vehicles are trying to enter in the same road ($\sum_i S_i > Q(\rho^*)$), there is a sort of mutual embarrassment which decreases the capacity of the outgoing road ($R_3' \leq g(\cdot) < Q(\rho^*)$);
- the $\min(R_3, \cdot)$ to be guarantee to remain in a subregion of (7.24).

We may now maximise the passing flow with any priority factors described in the previous paragraph with

$$Q(\bar{\rho}_i) \in \begin{cases} [0, S_i] & i = 1, 2 \\ [0, R_3'] & i = 3 \end{cases}$$

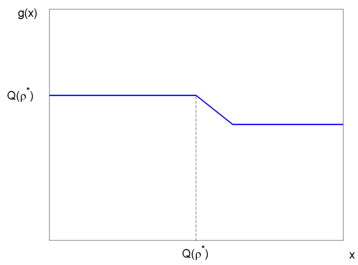
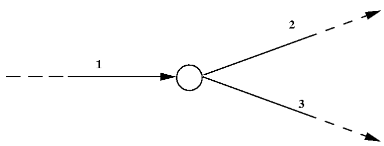


Figure 7.24: Possible shape of a g-"capacity drop function".

instead of (7.24).

This redefinition of the reception capacity is a correct expression of the *capacity drop phenomenon*: if too much drivers are trying to access the same road (a traffic jam is occurring), the output flow will be lower than the maximum achievable flow ($Q(\rho^*)$). As we will see in the simulation experiments, this modification has significant consequences on the system behaviour.

The diverge junction



If we know that the proportion of the vehicles from road 1 wishing to enter road 2 is α then the flow leaving the first road to enter the second is

$$F_2 = \min(\alpha S_1, R_2)$$

and the flow entering the third road is

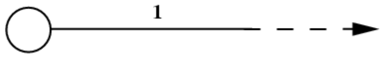
$$F_3 = \min((1-\alpha)S_1, R_3).$$

After having obtained the flows, the new densities at the borders of the junction are chosen such as

$$\begin{aligned} Q(\bar{\rho}_1) &= F_2 + F_3 \\ Q(\bar{\rho}_2) &= F_2 \\ Q(\bar{\rho}_3) &= F_3. \end{aligned}$$

In a practical point of view, the proportion α can be computed if we use different classes based on the origin–destination paths.

The origin junction



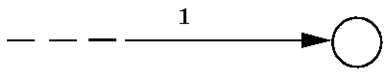
The origin junction is used to inject vehicles into the network. To model the origin junction we introduce a virtual cell of length Δx used as a buffer to store the flow wishing to enter the road but not yet able to do it. This can be the case if the road 1 is too much congested. If the flow wishing to enter the network is described by the function $D(t)$, then the flow F entering the road is

$$F = \min(S(\rho_b), R)$$

where ρ_b denotes the density in the buffer and R the receiving capacity of the outgoing road. The dynamic of the density in the buffer is then

$$\frac{d\rho_b}{dt} = \frac{1}{\Delta x}(D(t) - F).$$

The end junction



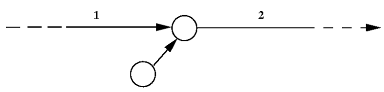
The end junction is used to represent a final destination of some vehicles. When the vehicles leave the last cell before the junction, they definitively disappear from the system. For the modelling of the end junction, we may assume that the road is not influenced by what is after the junction, i.e. by imposing a Neumann condition ($\frac{\partial \rho}{\partial x} = 0$) at the end of the road. Imposing a Neumann condition correspond to have

$$F = Q(\rho_f)$$

where F is the flow leaving the last cell and ρ_f is the density in this last cell. In some case, it may be useful to impose a upper bound F_{max} to the flow. For example, we may know that in the "real world" the vehicles leaving the system via the end junction enter an already congested road which may only accept a maximum flow F_{max}. Thus, in the simulator, the flow leaving the last cell of the road is

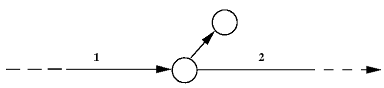
$$F = \min(Q(\rho_f), F_{\text{max}}).$$

The on-ramp junction



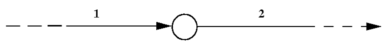
The on-ramp may be seen as a combination of a merge and an origin junction with a very short road between the two. The modelling of the on-ramp junction is a direct consequence of the choices made for the modelling of the merge and origin junctions.

The off-ramp junction



The off-ramp may be seen as a combination of a diverge and an end junction with a very short road between the two. The modelling of the off-ramp junction is a direct consequence of the choices made for the modelling of the diverge and end junctions.

The change of road property junction



The properties of one road (number of lanes, maximum speed,...) are reflected by the function $Q(\rho)$ used on this road.

For example, if we consider two roads (1 and 2) with different properties, the roads will have different flow function ($Q_1(\rho)$ and $Q_2(\rho)$). If we connect these two roads, the flow F between the two roads is simply

$$F = \min(S_1, R_2)$$

where S_1 is the sending capacity of first road computed using Q_1 and R_2 the receiving capacity of the second road computed using Q_2. This is the most natural model based on the assumptions already made for the merge junction (maximisation of the passing flow).

7.8.2 The implemented simulator

General overview

A simulator in C++ for an arbitrary road network based on the model (7.20) and the junction modelling described in Section 7.8.1 has been developed. The numerical scheme used in the simulator is the "extended" Daganzo scheme introduced in Section 7.8.1. The distinction between classes is based upon origin–destination paths. Only one main $Q(\rho)$ function may be specified by the user for the whole network. By convention, this $Q(\rho)$ function represents the flow–density relation for a three lanes road without additional speed limitation. If the user specifies a different number of lanes and/or a speed limitation for an arc, the flow–density relation used for this arc is constructed from the main $Q(\rho)$ relation by the following transformation:

$$Q_i(\rho) = \rho \min(v_{\text{max}}, \underbrace{\frac{i}{3}\,\frac{Q\left(\frac{3}{i}\rho\right)}{\rho}}_{U_i(\rho)})$$

where i is the number of lanes and v_{max} the maximum allowed speed.

The simulator is constructed in order to be able to study the effect of various traffic management and control strategies. It means that some properties (maximum allowed speed, the maximum flows leaving the origin junction,...) may be updated "on line" during the simulation.

The user inputs

The user may encode the topology of the network used in the simulator using a graph editor (see Fig. 7.25). He can also specify, using the graph editor, some parameters such as:

- the length of the arcs ;
- the number of lanes of the arcs ;
- the maximal allowed speed on the arcs ;
- the type of the junction (in the case of a node with one incoming and one outgoing arc, there exist different possibilities) ;

- the maximal allowed output flow rate in case of an end or off-ramp junction.

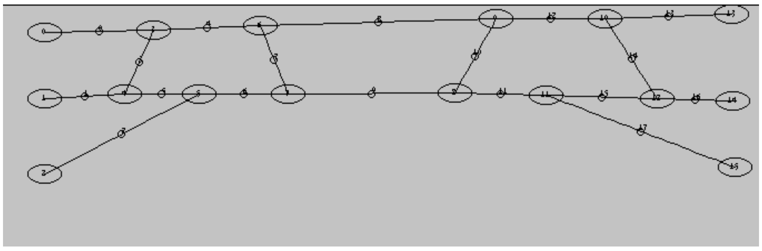


Figure 7.25: The encoding of the network topology.

The other parameters are specified using text files:

1. The different classes of vehicles considered by specifying the origin–destination paths. If the user encodes only the origin and the destination, the shortest path (in length) will be assumed.
2. The flow demand at the entrances (origins and on-ramps) which may by time varying.
3. The remaining parameters:
   - the length of the simulation;
   - the time step Δt used in the numerical scheme;
   - the length step Δx used in the numerical scheme;
   - if the capacity drop phenomenon is active;
   - if some noise with zero mean is added to the flow described by the user.

The simulator outputs

The evolution of the density–flow–speed on each arc are plotted and displayed (see Fig. 7.26) during the simulation. Moreover the data are written in text files for possible post-processing.

Different costs are considered in the simulator:

- the total time spent by all the drivers on the network which is expressed in [veh h];
- a time cost which includes both the nominal cost and the overtime cost (see Section 7.7.1)

$$\frac{dJ_T}{dt} = \sum_{a\in A} \int_0^{d_a} \left(C_{nom}\rho_a(x,t) \left[\frac{u_a(x,t)}{u_a^{max}}\right] + C_{ot}\rho_a(x,t)\left[1 - \frac{u_a(x,t)}{u_a^{max}}\right]\right) dx.$$

- the pollution cost (see Section 7.7.2)

$$\frac{J_P}{dt} = \sum_{a\in A} \int_0^{d_a} C_a^P(u_a)u_a\rho_a dx.$$

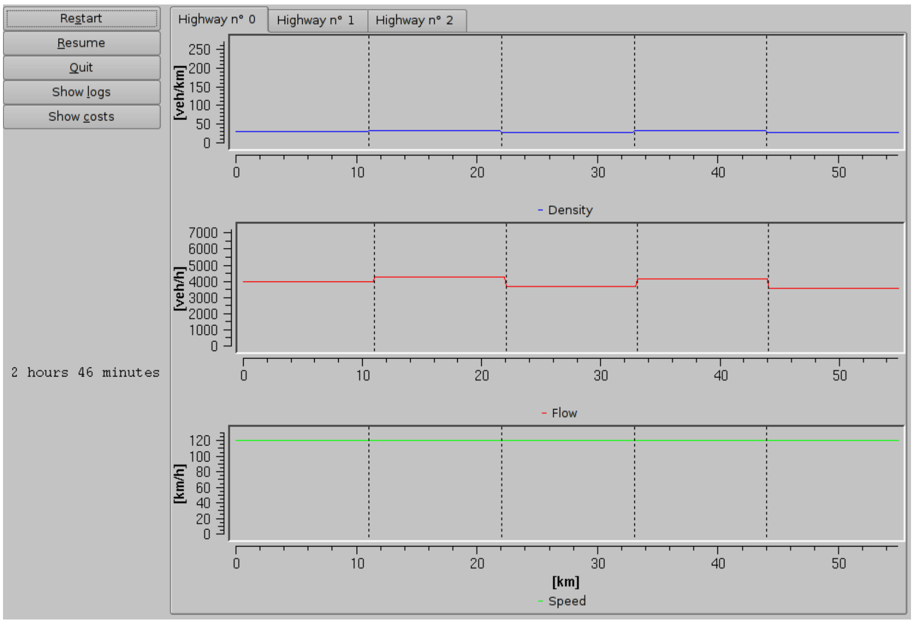


Figure 7.26: The evolution of the density–flow–speed during the simulation.

In the simulator, some buffers are also used to represent the vehicles which are not yet able to enter into the network. Assuming that the speed of the vehicles in those buffers is equal to zero, we can express the contributions to the costs due to the presence of the vehicles in a buffer as

$$\begin{aligned} \text{Time cost} &= \int_0^T n(t) C_{ot} dt \\ \text{Pollution cost} &= \int_0^T \lim_{u \to 0} C_a^P(u) u \quad n(t) dt \end{aligned}$$

where $n(t)$ is the number of vehicles in the buffer and T the length of the simulation.

The values of the total costs are updated and displayed during the simulation. Moreover the cost rates are graphically displayed allowing the user to see the effect on the total costs of the different events (see Fig. 7.27).

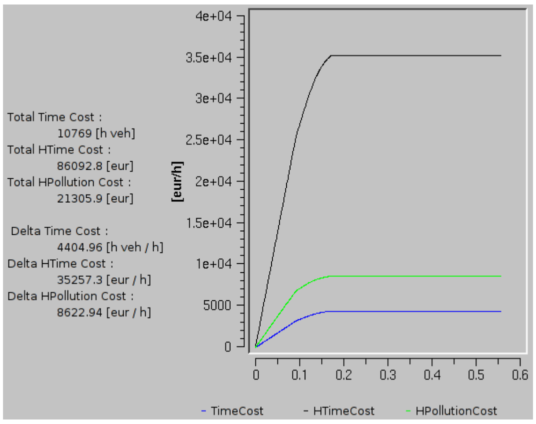


Figure 7.27: The evolution of the costs during the simulation.

7.9 Simulation experiments

7.9.1 Model validation

Some simulations may be carried out to check the validity of the simulator. For example, using the data collected by the MET on the E411, we may evaluate a possible speed–density relation $U(\rho)$. These data, collected on a minutely basis at ten stations distributed between Daussoulx and Overijse for a period of one month between October 1, 2003 and October 31, 2003, have lead to the functions $U(\rho)$ and $Q(\rho)$ represented in Figure 7.28.

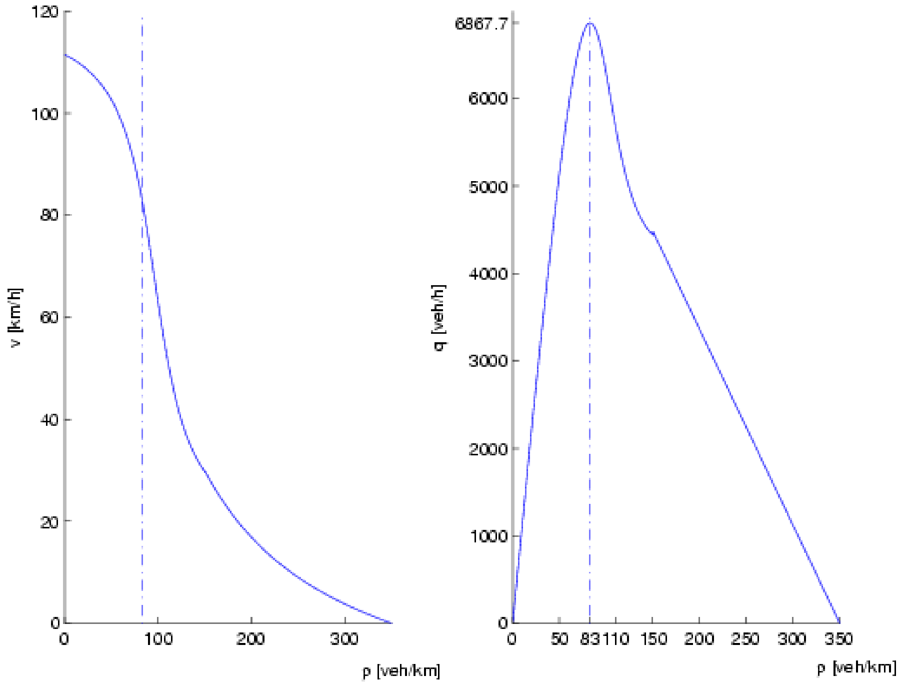


Figure 7.28: The functions $U(\rho)$ and $Q(\rho)$ used for the E411 .

On the basis of the flow measured after and before the access ramps, we may estimate the input flows at the entrances. In Figure 7.29 the simulation of the traffic evolution on the E411 between Daussoulx and Louvranges is represented. The simulated density is represented in red and the measured data in blue. Each graph represents the density in function of the time at a particular position. These particular positions correspond to the position of the counting posts (post 25: Daussoulx and post 11: Louvranges). We can notice that the simulated density is close to the measured one.

We can also simulate the occurence of a traffic jam at the entrance of Brussels. If we increase a little the flows at the on-ramps, we may temporary have a flow at the end of the E411 highway wishing the enter the ring too high for the ring reception capacity. We will then have a traffic jam growing at the end of the E411 highway. This is illustrated in the Figure 7.30 where the density in function of the time a few kilometres before the Leonard crossroad is represented.

7.9.2 A case study

In this simulation, the influence of the capacity drop phenomenon is illustrated. This phenomenon is modelled using the function $g(x)$ as explained in Section 7.8.1. For this simulation, we use a simple

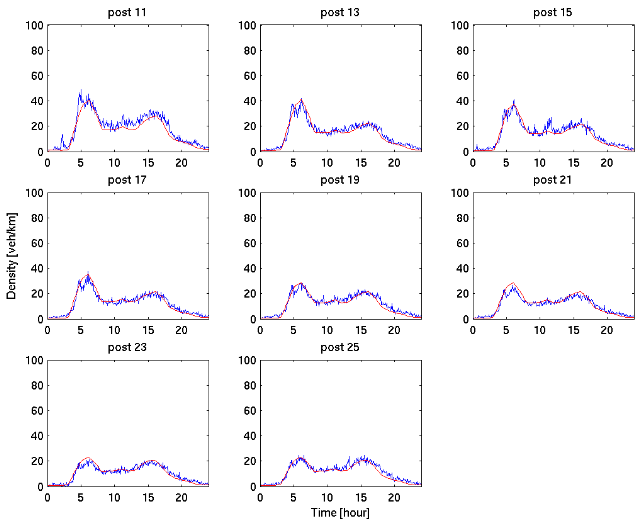


Figure 7.29: The evolution of the traffic on the E411 on October 10th 2003.

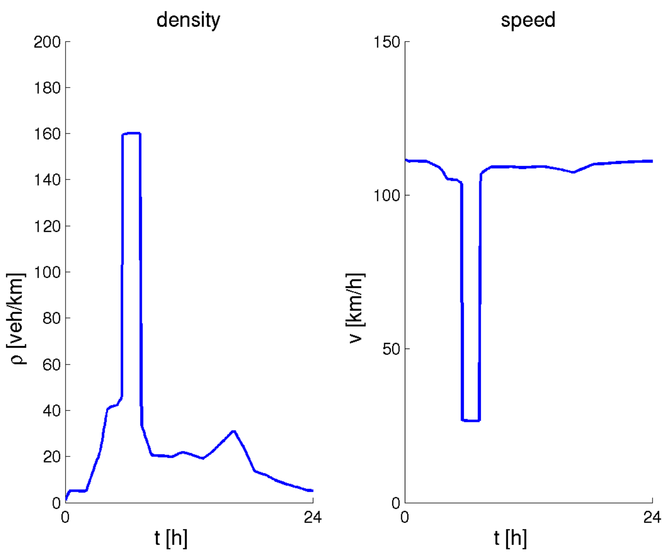
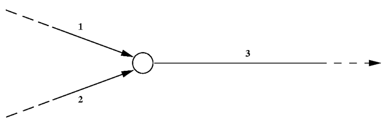


Figure 7.30: The occurence of a traffic jam at the end of the E411.

network consisting of two roads merging in one.



The function $Q(\rho)$ used has a maximal value of $Q(\rho^*) = 2880$. As initial condition, we take 1400 [veh/h] for the flows on the two incoming roads and 2800 on the outgoing one. This is an admissible state for (7.24) and it is optimal for the criterion presented in Section 7.8.1. After a while, a temporary increase of flow up to 1550 is added at the beginning of the first incoming road (see Fig. 7.31). The

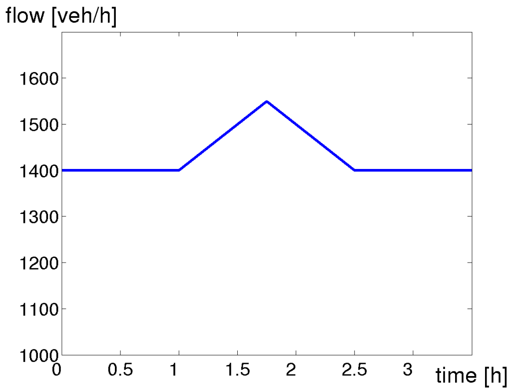


Figure 7.31: The flow at the entrance of the first incoming road.

sum of the two incoming flows (1400+1550=2950) is too high for the outgoing road so a traffic jam will occur. We may now distinguish two cases:

without the capacity drop representation : the traffic jam will grow and, when the incoming flow on the first road will finally go back to 1400, the traffic jam will decrease to finally disappear (see Fig. 7.32 where the traffic state on one of the incoming roads and on the outgoing one are represented).

with the capacity drop representation : because of the presence of the traffic jams in front of the junction, the reception capacity of the outgoing road will drop down to 2736 (=$0.95Q(\rho^*)$). The traffic jam will grow and, even after that the incoming flow on the first road has gone back to 1400, the sum of the incoming flows is too high for the new reception capacity of the outgoing road (1400+1400 > 2736). As we can see in Fig. 7.33, the traffic jam will never stop of growing.

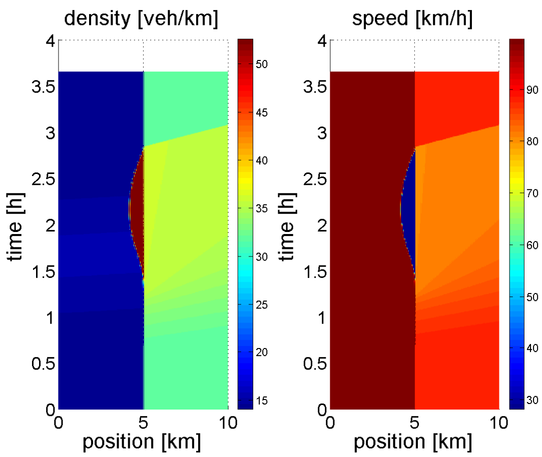


Figure 7.32: The evolution of the traffic state in absence of a capacity drop representation.

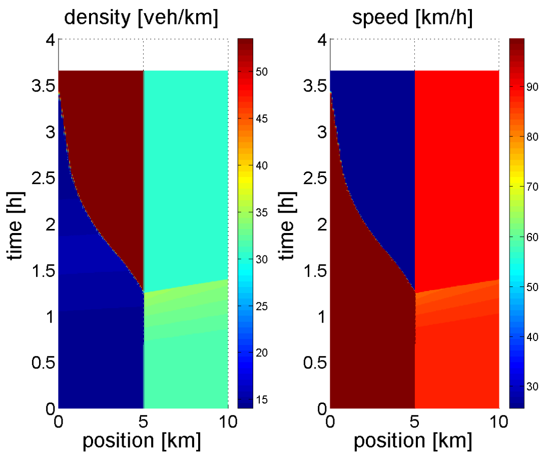


Figure 7.33: The evolution of the traffic state in presence of a capacity drop representation.

7.9.3 Traffic management and control studies

It may be useful to use some regulation strategies such as speed limitation or ramp metering in order to improve the usage of a road network. As explained in Section 7.4, modifying the maximum velocity allowed on some arcs may improve the maximal passing flow by changing the preferential road of the drivers. But even if the drivers have only one possible path, some regulation strategy may be useful to improve the total sustainable cost. In a simple example (see Fig. 7.34) constituted of two incoming roads and one outgoing, we can illustrate the effects of some ramp metering or speed limitation strategies.

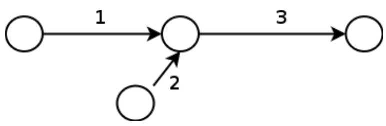


Figure 7.34: A simple network constituted of two incoming roads and one outgoing.

The demand at the entrance of road 2 is constant and equal to 500 veh/h while the demand at the entrance of road 1 is a function of time (see Fig. 7.35). The capacity of each road is equal to 2880 veh/h.

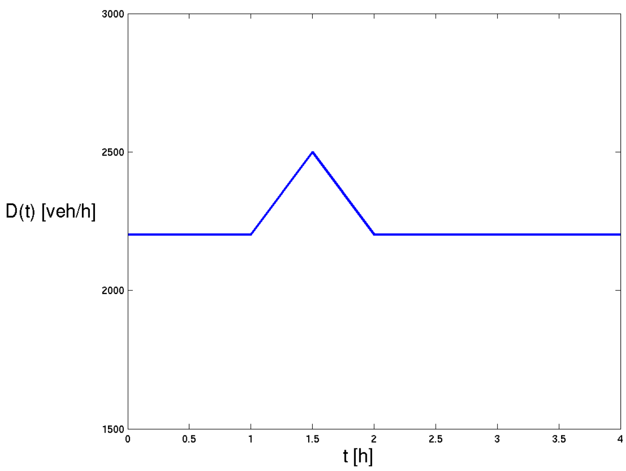


Figure 7.35: The demand at the entrance of road 1.

At the beginning, all the flow may pass through the network (2200 + 500 < 2880). When the flow entering in road 1 is too high, there is a saturation at the entrance of road 3 (2500 + 500 > 2880) and the reception capacity of road 3 drops down to 2600. This new reception capacity is always smaller than the total demand at the origins. A permanent traffic jam will thus occur even when the demand at the entrance of road 1 returns to 2200. See Fig. 7.36 for the evolution of the density and the speed on road 1 and 3 and Fig. 7.37 for the evolution of the cost rates. In Table 7.15, we report the total costs at the end of a 4 hours simulation.

Different regulation strategies may be used in order to prevent this permanent traffic jam. For example, we may use some ramp metering to limit the flow leaving the entrance of road 1 to 2350 veh/h. With this restriction, the maximum flow trying to enter road 3 is lesser than the reception capacity of road 3 (2350 + 500 < 2880) and no saturation may happen at the entrance of this road. See Fig 7.38 for the evolution of the density and the speed and Fig. 7.39 for the evolution of the cost rates. As we can see in Table 7.15, with this regulation strategy, both the time cost and the pollution cost are smaller than without control.

As we have seen, it is important to prevent the saturation at the entrance of road 3. This can also be done with a speed limitation on road 1 which reduces sufficiently the capacity of this road. This speed limitation will result in a higher density on the first road for an equal or smaller flow. Instead of storing the vehicles at the entrance, we store them on the first road. With a speed limitation at 50 km/h on road 1, we obtain the results represented in Fig. 7.40 and Fig. 7.41. As we can see in Table 7.15, with this regulation the time cost is higher than with a ramp metering strategy but the pollution cost is less important.

	Time Cost [€]	Pollution Cost [€]	Total Cost [€]
No control	17708	1806	19514
Ramp metering	9364	1742	11106
Speed limitation	14015	1695	15710

Table 7.15: The total costs after a 4 hours simulation.

Without control, the total demand is not achieved (336 vehicles are waiting in the buffer of the first entrance for a total demand of 10950 vehicles). With any of the two control methods used, the total demand is achieved (no vehicle in the buffer).

In these simulations, no control was applied on the second road. Of course we could envisage to establish a control on the second road in order to spare the annoyances between the different drivers.

We have used here some “simple” control strategies such as fixed ramp metering or fixed speed limitation, but it is clear that we could improve the sustainable cost by considering variable control strategies. By example, it is not necessary to limit the speed on the first road when the total demand at the two entrances is smaller than the reception capacity of road 3.

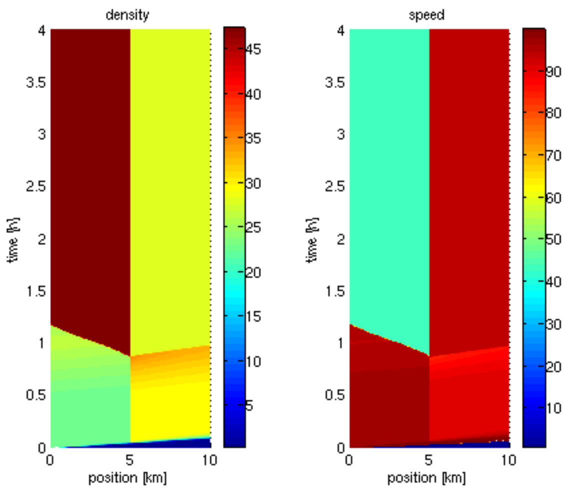


Figure 7.36: The evolution of the density and the speed on road 1 and 3 without control.

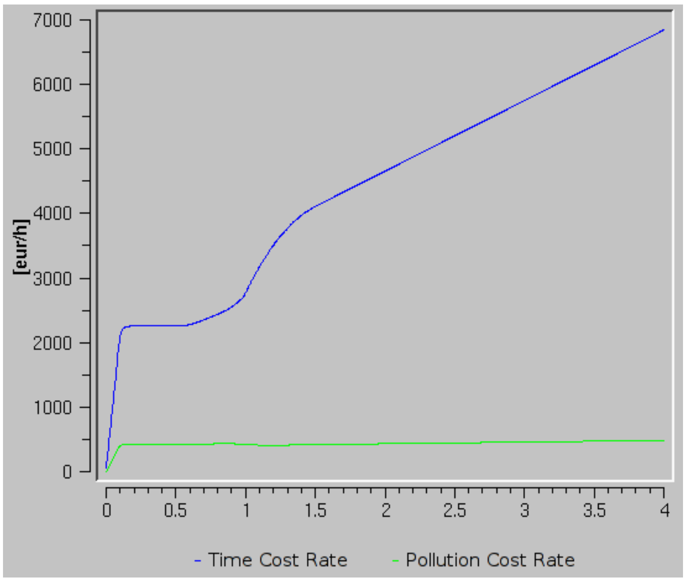


Figure 7.37: The evolution of the cost rates without control.

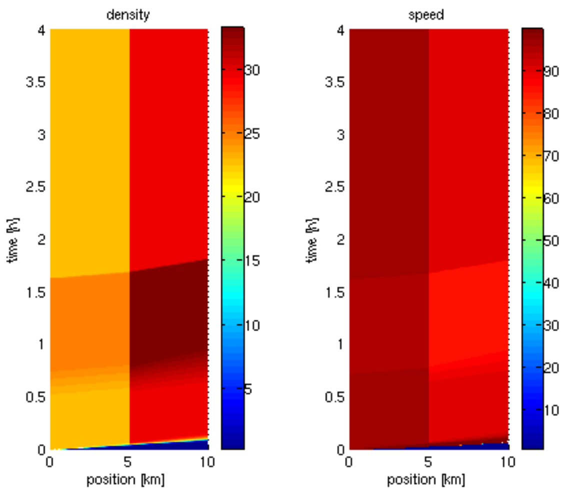


Figure 7.38: The evolution of the density and the speed on road 1 and 3 with ramp-metering.

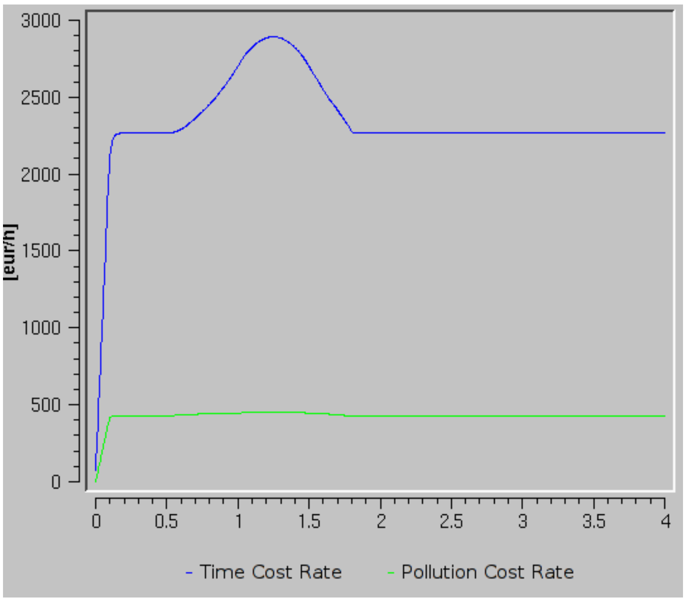


Figure 7.39: The evolution of the cost rates with ramp-metering.

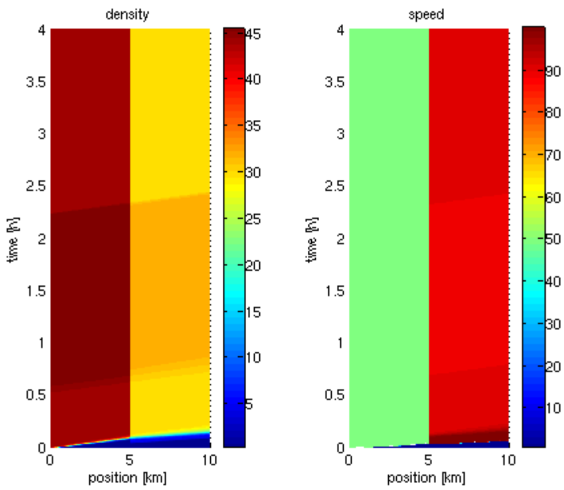


Figure 7.40: The evolution of the density and the speed on road 1 and 3 with speed limitation.

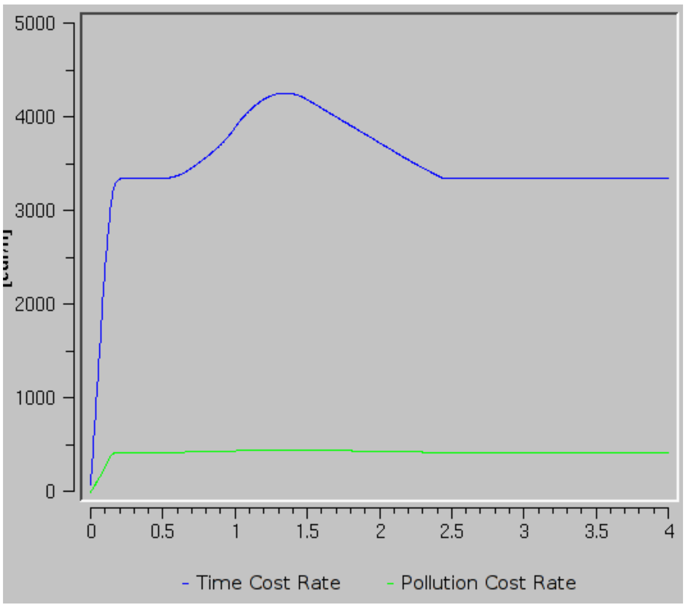


Figure 7.41: The evolution of the cost rates with speed limitation.

Part III

Future prospects and future planning

The results of this research can be split up in the following categories

(1) Scientific knowledge

The research will result in extensive new scientific knowledge that will be presented on congresses and will be published in journals. New knowledge and experiences can certainly be found within:

- The processing of traffic data from traffic detectors and new advanced traffic detection systems like GSM ...
- The set-up of a dynamic model on a regional scale.
- The mixed use and the tuning of macroscopic and microscopic dynamic traffic models.
- The dynamic OD estimation.
- The set up and the implementation of a sustainable cost function for the evaluation of dynamic traffic patterns.
- The development and calibration of dynamic models can be linked to the static equilibrium approach. The marginal cost function for congestion that is one of the fundaments of the external pricing theories can be controlled from a dynamic point of view.
- The design of new control strategies for ATMS.
- The application of optimisation within traffic research.

(2) Practical use of results for Belgium

The results of this research will be of importance for the Belgium country. First of all the Belgium and federal road Administration can use the results to improve their transportation system.

- A lot of new ATMS will be installed in the near future. The new Control strategies can be implemented immediately in these systems.
- The project will set up a model of a part of the Belgium road network. This model can be use for other purposes by the road Administrator.
- The road administrator asks new dynamic traffic models. This project forms a big experience for these future developments and improves the practical knowledge in Belgium of dynamic traffic flow modelling.

Secondly the members of the user comittee will be involved in the transfer of technology and knowledge. Training sessions are planned and a web site can spread information towards the interested public. There is also a possible knowledge transfer towards university spin-offs.

The implementation of this research needs a big use of practical data so that there is a potential collaboration with industrial partners like Siemens, Traficon, Tritel, TNO, Abay, ...

Part IV

Annexes

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