Hyder

The Highways Agency

Motorway Travel Time Variability

Final Report

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Motorway Travel Time Variability

Final Report

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CONTENTS

EXECUTIVE SUMMARY

The headline achievement of this study is the development of a model which accurately forecasts travel times and Travel Time Variability (TTV) on motorways where flow breakdown occurs in congested conditions. This fills a critical gap in current traffic modelling and appraisal techniques, which are unable to reliably model flow breakdown and realistically reproduce journey times and TTV in congested conditions. The detailed understanding of flow breakdown gained in this study could also facilitate enhancement of the methods currently used for managing congestion on the motorway network.

Background

Travel Time Variability (TTV), colloquially referred to as "reliability", is receiving increasing attention in a number of countries, and is becoming an important aspect of scheme appraisal. TTV arises from two main sources: incidents (typically accidents or vehicle breakdowns) which reduce carriageway capacity, and congestion.

WebTAG Unit 3.5.7 "The Reliability Sub-Objective" sets out the approach to the appraisal of TTV that is currently recommended by the Department for Transport (DfT). It notes that for inter-urban roads when flows are below capacity, TTV arises mainly from incidents. For appraisal of TTV in this situation INCA is the DfT's recommended approach. However, in the case where the flow exceeds capacity and flow breakdown occurs, current transport modelling and economic appraisal techniques are not adequate for predicting either the excess travel times or the TTV caused by flow breakdown. These are major omissions which are likely to lead to the mis-estimation of benefits for schemes relating to congested sections of the highway network where flow breakdown occurs, and the consequent mis-specification of scheme features and priorities for congested links.

The research described in this report has the objective of addressing this gap in modelling and appraisal. It focuses on motorways in particular, which are susceptible to large variations in travel time under congested conditions, but its findings are potentially also applicable to other roads. It has provided an understanding of the main causes of excess journey times and TTV due to congestion. This will facilitate the development of management strategies and schemes designed to reduce excess travel times and TTV.

The project follows on from a previous study in 2003 for DfT during which a substantial dataset was collected for an 11 km long highly congested section of the M6 motorway northbound between Junctions 8 and 10A near Birmingham. The present study was commissioned with a view to analysing the data and building appropriate supply models that explain the observed pattern of excess travel times and TTV, and which could be used for prediction and appraisal. The finalised databases output from the study are an important resource for analysing motorway congestion and are described in detail in (Hyder, May 2010).

Study Findings

The study has provided a detailed understanding of flow breakdown and its role in affecting journey times and TTV. The term "flow breakdown" is widely used to describe the sudden change in the flow of traffic on motorways from smooth and fast to stop-go. Two types of flow breakdown have been identified:

- "Shockwaves", which occur at specific seed-points (e.g. locations with gradients, or when a driver suddenly applies the breaks) and are of relatively short duration; and

- "Standing waves" which typically arise at motorway merges, and last for extended periods (6 hours or more in the study location).

This study has found that standing waves are the main cause of excess travel time and TTV on the congested section of the M6 that was studied, and by implication on congested sections of the highway network in general.

Flow breakdown is a stochastic phenomenon which is likely to occur in periods where demand is relatively high. Delay and TTV caused by 'standing wave' flow breakdown in the case of M6 is dictated more by supply than demand variability. In particular, the time at which flow breakdown occurs is critical in determining the level of delay and TTV. Even though the underlying probability of breakdown in any 5-minute period is low, within a prolonged period of high demand flow breakdown at some time becomes almost certain. The impact of flow breakdown depends strongly on when it occurs in relation to the profile of demand over the day. The importance of an accurate estimate of day to day variation in demand should also be emphasized, as TTV is very sensitive to this.

The main achievement of the study has been to develop a model (the "Aggregate Model") which replicates the observed variation in mean travel time and TTV over the course of a day for the above dataset. The model adopts a simulation approach at a relatively aggregate level, working in five-minute intervals. The model validation exercise has shown that the Aggregate Model can reproduce the phenomenon of flow breakdown and its effects on the profiles of both mean travel time and TTV, with a high degree of accuracy. In the case of mean travel time the difference between weekdays is reproduced well, as is the pattern of travel time over the breakdown period. In the case of TTV, both the pattern and scale of the change in TTV is reproduced.

The Aggregate Model has four main components:

- the mean profile of demand for traffic entering the modelled network, together with its variation from day to day;

- a simple dynamic assignment model, in which the network is represented by a relatively small number of links;

- a "Breakdown Probability Function", which forecasts the probability of flow breakdown occurring at a given time; and

- a "Queue Discharge Function", which forecasts the rate at which the queue discharges once flow breakdown has occurred.

The Aggregate Model offers the potential for the prediction of both TTV and the delays caused by flow breakdown – something which conventional traffic models are not able to do. It is not complex and therefore could be applied at relatively low cost. Its ability to reproduce daily profiles of average travel time at the level of accuracy seen in this study using data from the M6 suggests that it has an important role to play in the analysis of flow breakdown and initiatives to reduce its impact. At present the results relate to a small section of motorway comprising three merge locations. Additional data (on demand and propensity for breakdown) will be required for new locations, but the data requirements are generally compatible with that which can be provided by MIDAS. It should also be possible to use the model for improving operational strategies for dealing with, and mitigating, the effects of breakdown.

Recommendations

Further work would be required to develop the Aggregate Model into a tool for appraising the full range of possible interventions that are designed to improve conditions on congested motorways and other roads. For this, the model parameters would need to be calibrated for a wider range of flow breakdown locations, and the Aggregate Model integrated with the general highway scheme appraisal process. The resulting model would provide the tool for:

- assessing network performance to identify the need for intervention;
- undertaking operational analysis to develop appropriate interventions; and
- the appraisal of any such interventions.

The output of this further work would be a procedure which would greatly enhance the ability to develop cost effective initiatives to efficiently manage increasing congestion on the motorway and primary road networks and to minimise its economic impacts in terms of both travel times and TTV.

In the longer term, it would also be appropriate to develop an integrated tool for dealing with both of the predominant causes of motorway TTV – i.e. flow breakdown and incidents. This would involve accepting the need to model the profile of demand within a dynamic assignment procedure.

In the short term, the following work packages are recommended:

WP1 – Develop a strategy for wider implementation of the Aggregate Model in the modelling and appraisal process;

WP2 – Implement the current Aggregate Model for one or more other locations including the development of appropriate model parameters from MIDAS data for different flow breakdown location types e.g. diverges and merges at junctions with lane drop and gain;

WP3 – Assess the impact of detailed dynamic 5-minute period modelling as in the Aggregate Model on the calculation of incident delays and related TTV;

WP4 – Review operating algorithms for motorway management measures such as ramp metering to assess how the findings of this study could enhance their performance in ameliorating flow breakdown due to congestion. Also assess how the findings of this study should be used to inform the Highways Agency's motorway management systems.

1 INTRODUCTION

1.1 Travel Time Variability

An essential part of project and policy appraisal is an estimate of the associated travel time savings. For the most part, transport models assume that, given the demand and supply, travel time is known with certainty. As users of transport systems are well aware, in practice this is not so. Transport systems are subject to unreliability, and transport decisions cannot be made on the basis of certainty.

When discussing reliability in the transport context, the essential concept is the variability of travel time. For this reason, there is some tradition of only using the word "reliability" informally, and referring more accurately to "Travel Time Variability" (TTV). This will be the approach taken in this report.

It is essential, at the outset, to be clear at what level we are considering TTV. For example, travel time on the highway network is affected by congestion, and the time taken to travel between two points is generally longer during peak periods than it is in the middle of the night. This leads to the conclusion that TTV should not take account of variations in travel time that are inherently predictable, even if particular travellers are not adequately informed. We are essentially concerned with unexpected changes to the normal (expected) travel time, at a particular time of day.

We can identify three principal causes of TTV. The first is **incidents**. These are randomly occurring events, typically leading to a loss in capacity: on the highway side, they could be traffic-related (e.g. vehicle breakdown, collision, or simply an illegally parked car or lorry), or they could be infrastructure-related (e.g. emergency roadworks, a burst water main, traffic signal component failure).

The other two reasons fall under the heading of **"Day-To-Day Variability"** (DTDV). Following the work of Arup (Arup, December 2003), there are separate effects on the supply and demand sides:

- On the demand side we have random (unpredictable) variations in demand, which, in as far as they lead to supply effects, will affect travel times;
- On the supply side, we have other random operating conditions (e.g. weather, random vehicle type composition) which would lead to different travel times even if the overall level of demand was identical.

Arup expressed this as the following "equations":

TTV = DTDV + Incident-related variability

where DTDV = Demand-related effects + Capacity-related effects

(although we now think that "supply-related" rather than "capacity-related" would be more appropriate for the last term).

The Eddington report (Eddington, 2006) recognised reliability as a key issue for transport policy and appraisal. In Chapter 1.2 of the report, it was noted that:

In addition to the importance of costs and journey time, journey reliability also matters. This is supported by survey evidence and economic analysis. Travellers and business want to know not only whether a particular journey will take 30 minutes on average but whether it will take 30 minutes every day, rather than 30 minutes most days and 60 minutes once a week[…].

Congestion on the network, failure to maintain assets and incidents can all affect the reliability of a journey. […]. For business, if materials or workers do not arrive on time, this can create bottlenecks and delays to production processes or result in the loss of perishable goods and service contracts. […] As a result, unreliability can also cost business in terms of contingency measures that need to be put in place.

In order to integrate reliability into our transport models, we need to know a) how changes in reliability will impact on demand, and b) how, for a given policy, the outturn reliability will be affected by the level of demand. These are the key supply and demand relationships.

1.2   Previous Arup Work

As part of its ongoing research into TTV, the DfT awarded a contract in 2001 to Arup, in collaboration with John Bates, John Fearon and Ian Black. Although the remit of the study did cover work on incidents and on wider demand issues, the focus was on DTDV, and for motorways in particular.

Arup identified as critical the issue of "transient excess demand" – short periods when capacity is exceeded, leading to queueing. On theoretical grounds, this was felt to be a major source of TTV, at least on motorways. It was considered essential to adopt a dynamic analysis, which allows for lagged effects of demand in previous "time-slices" whenever queues arise. This requires information about the temporal profile of demand.

Making progress was hampered by the fact that most existing network tools do not adequately represent the profile of travel time, and, more importantly, by the general lack of understanding of traffic phenomena under conditions of flow breakdown. In addition there are merging and weaving issues particularly where motorways merge, and significant diverge problems, such as when capacity problems on the slip road lead to tailback on the main carriageway, or even back to an upstream merge.

Given the importance attached to a methodology for predicting TTV on motorways which experience transient excess demand, it was apparent that only very limited progress could be made with the analysis of existing datasets (as had been the intention of the Brief for the Arup study), so that there was an urgent need for new data collection. Noting that the aims of the study were limited to modelling a small number of successive motorway links, rather than the full complexity of a general network, Arup proposed collecting a year's data for an 11 km section of the M6 (northbound Junctions 8 to 10A).

However, as a result of the timescale involved in the data collection, the priorities for the remainder of the study were altered. It was accepted that it would not be possible to analyse the full dataset within the project. Prototype models were developed based on analysis of the first month's data only. One of these models was a microsimulation model using S-Paramics, and the other was an "aggregate simulation" model.

The first recommendation in the Arup final report (Arup, December 2003) was as follows:

The extensive data collected on the M6 should be fully analysed, and used to validate the existing micro-simulation and aggregate simulation models by examining longer periods, other days of the week and seasons of the year.

1.3   Flow Breakdown

The term "flow breakdown" will be widely used in this report and is generally found in the literature (see, for example, the Case Study for the Controlled M25 Motorway on the DfT

Website, and the description of the "Network Active Traffic Management Supervisory Sub-System" on the Highways Agency website). Nonetheless, it is difficult to find a precise definition. It is generally associated with the sudden change in the flow of traffic on motorways from smooth and fast to stop-go. In a pioneering study, Hounsell et al (TRL & Southampton University, 1992) investigated a number of reported cases of flow breakdown on motorways, but still did not produce an exact definition. For identification purposes, he proposed criteria for "speed breakdown" and "flow breakdown" as follows (op. cit., Appendix A):

Speed Breakdown *The occurrence of 'speed breakdown' on a section of motorway was identified in this study with reference to the speed-time profiles for traffic in lane 3, where breakdown was invariably initiated. …. Speed breakdown was taken to correspond to a rapid reduction in speeds in excess of some 20-30 km/h, resulting in residual speeds of below 80 km/h. This 'definition' proved more practical than a variety of more rigorous mathematical definitions which were investigated.*

Flow Breakdown *The occurrence and timing of flow breakdown was identified with reference to the minute-by-minute speed-flow plots, an example of which is given in Figure A2. This illustrates how, at 0703 hours, speeds and flows first started to decrease simultaneously, whereas earlier high flows (e.g. at 0657 hours) were not followed by a reduction in both speed and flow.*

The issue was discussed carefully in Section 8.2 of (Arup, December 2003), though, once again, no definition was given. Paragraph 8.2.4 stated:

The reduction in speed associated with "breakdown" is commonly interpreted as a movement to a different speed-flow regime. The TRLUN report (TRL & University of Newcastle, 2000) seems to imply a shift between the two portions of the classic backward-bending curve: CR338 (TRL & Southampton University, 1992) on the other hand seems to imply movement to a lower but parallel speed-flow relationship which appears to be associated with a reduction of 5-10% in capacity.

While flow breakdown may of course be associated with an incident, we are particularly interested in the case of merges, where the level of demand potentially increases. Section 4 discusses this critical issue in more depth.

1.4   The HA Brief

At the end of 2008, following discussions with the Highways Agency, a proposal was made by Hyder (as part of the Séligere consortium comprising Arup, Capita Symonds and Hyder Consulting), in collaboration with the John Fearon Consultancy, John Bates and Ian Black, with the aim of acting on the Arup recommendation for the analysis of the data (cited earlier), and ultimately developing a practical model, suitable for use in scheme evaluation and policy analysis, of TTV on motorways, that specifically allows for the phenomenon of transient excess demand, or flow break-down.

A short initial contract (subsequently referred to as "Stage 1") was let with the aim of demonstrating that the various components of the earlier Arup project were all still accessible and in functioning order. The data was retrieved and examined and no issues which would be significantly detrimental to the objectives of the research were detected. The S-Paramics model M6SIM was reviewed, and some adjustments proposed to bring it up to date with current practice. Key results documented in the Arup final report were reproduced to an acceptable level of agreement, and the "shell" program (M6FRAME) for generating a random set of trip matrices ("demands") and release profiles for simulated "days" was tested and found to be satisfactory. Finally, the aggregate simulation model was retrieved and reviewed in the light of

more recent work, particularly in relation to flow breakdown. The work was completed in March 2009.

As a result of the initial work, a further proposal was made to continue with Stage 2. It was recommended that the remainder of the M6 data, except for that relating to periods where MIDAS count and speed data is not available for a high proportion of links, should be processed and analysed. From this data "day type" groupings for modelling should be identified. All inputs required for model development, validation and operation should be produced. Also additional analysis should be undertaken to inform the understanding of flow breakdown and the resulting travel time variability.

It was proposed in the first place to concentrate on DTDV and only to come back to the contribution of Incidents at a later stage. In the event, it has not been possible to advance the work on incidents, and the main aim of the project has been the development of models of DTDV due to excess demand. In addition, however, the M6 data also provides the opportunity to develop models of delay in periods of excess demand. The lack of such models has caused problems in the appraisal of schemes for congested motorways such as the M25. This data plus the proposed DTDV model development provides an opportunity to efficiently address this issue. It was therefore proposed to pursue the development of such models in addition to models of DTDV.

The investigations relating to both demand-related and supply-related variability could proceed in parallel, once the necessary data had been processed into an appropriate format. Periods affected by incidents have merely been identified and removed from the data.

For the demand-related variability, it was necessary to analyse the data in order to detect "predictable" patterns of variation – essentially making use of some of the techniques discussed in Section 5. The aim was to identify a limited number of "day types", concentrating in the first place on school term time weekdays (provided the appropriateness of this category is confirmed by analysis) and then at a later stage going on to other day types. The identification of appropriate day types would be based on appropriate statistical analysis. The conclusions regarding mean and variance of the profile of daily demand constitute the key inputs to the M6FRAME and aggregate simulation models.

On the supply side, it was considered useful in the first place to determine the relevance of various parameters in the M6SIM micro-simulation model to flow breakdown. Sensitivity testing was suggested a number of areas, and the recommended change to the J10 merge ramp was to be implemented and tested.

Next, it was thought necessary to distinguish between the case of flow breakdown and the case of "normal" operation (even if under congested conditions). In Stage 1 we had emphasised the key role of flow breakdown in contributing to variability, and it was essential to demonstrate that the supply models (M6SIM, and the relevant component of the aggregate simulation) could reproduce the initiation and duration of breakdown and the consequent impact on travel time. In order to do this, it was proposed to select from the database a number of days in which breakdown occurs for analysis and model validation. While it might be of interest, in the first place, to examine the most extreme cases, we needed to be sure that the model would work well both for the average day and for extremes.

The Stage 1 work had confirmed the idea that flow breakdown itself should be regarded as a stochastic variable. The anticipated approach for each of the days being examined was to take the actual data (without demand variability) and use it for repeated runs of the supply model. In cases when flow breakdown does not occur, a more straightforward analysis for supply-related variability was thought to be possible, based on speed-flow data, as well as the ANPR data.

Once we were confident that we had an appropriate representation of both components of DTDV, the next stage of the process would be to carry out multiple runs of the systems, with the aim of comparing modelled TTV with observed TTV on the M6, in the absence of incidents.

If this was successful, we would have created test-beds for the examination of delay and variability under different demand conditions. Ultimately, the aim is to produce estimates of delay and variability, given demand and network conditions. The model(s) could then be further developed in two ways. The more traditional approach would be to transfer the model(s) with the same structure and parameters to different motorway locations. However, another possibility was that the results could be summarised into simple relationships that predict delay and TTV as a function of traffic demand or some other related variable (such as traffic flow or travel time). In either case, this would require validation against other situations (ie away from the particular stretch of the M6 which we have been examining). It may also be noted that since we have two main flow breakdown locations on the section of the M6 covered by our data (J8 merge and J10 merge), we have the potential, for example, to develop functions for the J8 merge and validate them for J10 merge.)

The Stage 2 work commenced, after an interval of 6 months, in October 2009.

1.5 Contents of the Report

The flow of work in the study is illustrated in Figure 1.1, which also indicates the Sections of the report in which each part of the work is described, as follows.

Section 2 describes the data, discussing the survey design and execution, the various problems encountered, and the outcome in terms of days of reliable data. Section 3 describes the analysis, including the identification of periods affected by incidents, the daily pattern of travel times, and hence a general description of TTV.

Section 4 sets the context for dealing with Flow Breakdown, commenting on recent work by TRL and drawing appropriate conclusions for the current study.

In Section 5, we describe the work done on the S-Paramics model, and the conclusions for demand-based TTV. As will be seen, despite considerable efforts, we concluded that the S-Paramics model would not deliver the results that we required for the analysis of TTV.

Section 6 provides an Outline of the Aggregate Model, with particular emphasis on the treatment of supply variability and flow breakdown.

Section 7 describes the Demand analysis, with the aim of providing appropriate input to the model in terms of Total daily demand and the profiles over the day, distinguished, as appropriate, by "day types"

In Section 8 we describe the Flow Breakdown analysis which was necessary to provide appropriate information for the Aggregate model, leading to the relationships for predicting breakdown (BDF) and the "queue discharge flows" (QDF). This is followed by Section 9, which provides a full description of the design and construction of the Aggregate Model, followed by testing and validation of the model.

Section 10 summarises the conclusions arising from this research and provides recommendations for taking the work forward.

Figure 1.1 Work on the Motorway TTV Project (and Sections of this Report)

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│  M6 Data Processing (2) │
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│   M6 Data Analysis (3)  │- - - - - │ Characteristics of Flow  │
│                         │          │     Breakdown (4)        │
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│   Microsimulation Model │          │ Aggregate Model Outline  │
│     Development (5)      │          │          (6)             │
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                   │  Demand Analysis (7)  │          │    Flow Breakdown      │
                   │                       │          │     Analysis (8)       │
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                                    │     Aggregate Model      │
                                    │ Development, Calibration &│
                                    │       Validation (9)      │
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                        │ Conclusions and Recommendations (10)│
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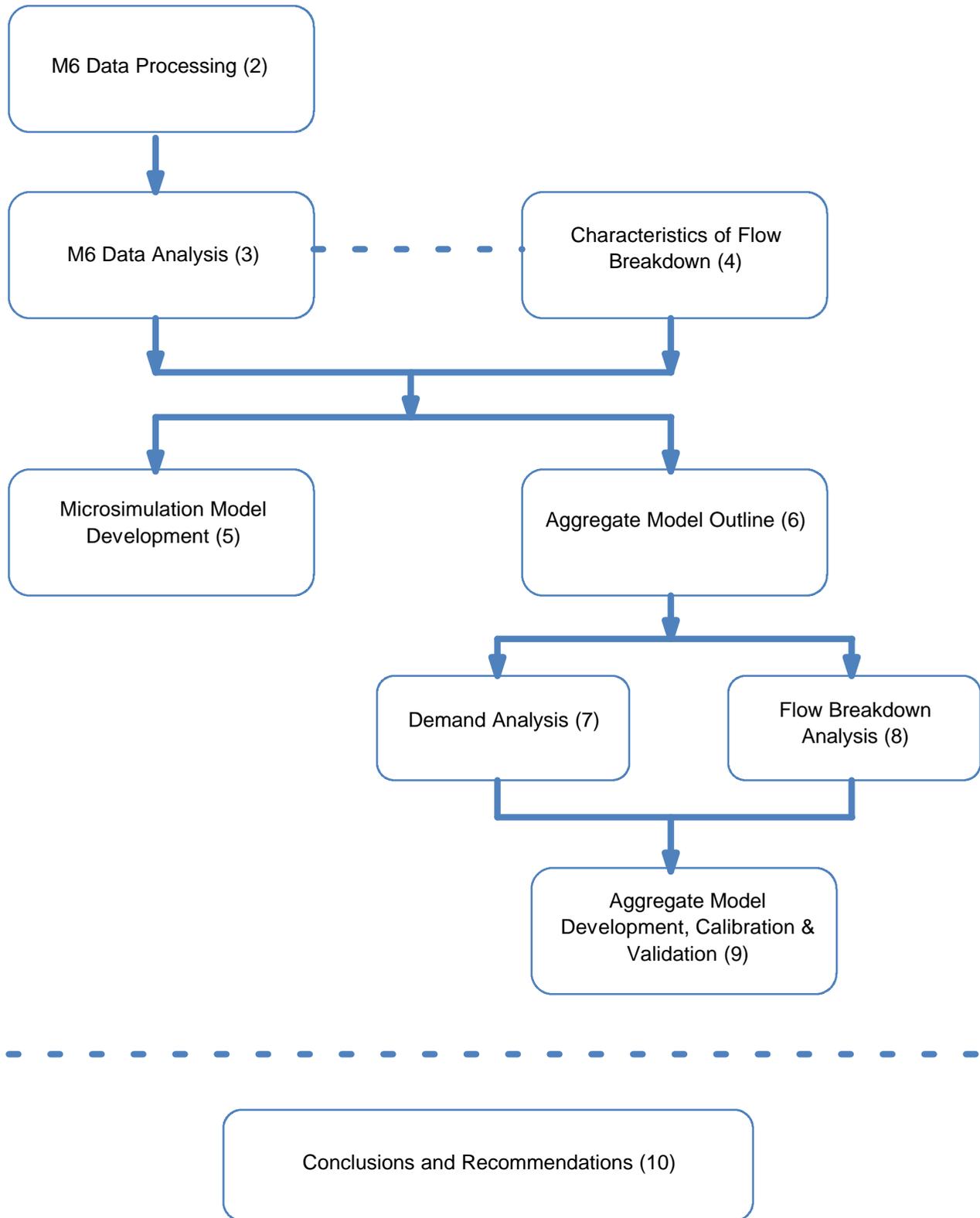
2 BACKGROUND TO THE DATA

2.1 Survey Outline

2.1.1 Background

The objective of the data collection undertaken on the M6 between Junctions 8 and 10A during 2003 was to provide a comprehensive dataset which would facilitate the development and validation of models which could be used to develop relationships to forecast motorway travel time variability (TTV) caused by excess demand. The survey design is described in detail in (Arup, December 2003).

2.1.2 Survey Location

Data collection needed to be on a section of motorway which has frequent flow breakdown due to excess demand, and is also covered by the MIDAS system, which would provide vehicle flow, speed and queue identification data. In view of these requirements the section of M6 between Junctions 8 and 10A was chosen. It was not possible to include sections of the M6 east of Junction 8 as at the time there were long term roadworks between Junctions 6 and 7 over which section the MIDAS loops were not operational.

To provide journey time data, six ANPR cameras were installed at the locations shown in Figure 2.1. These provide journey time data for northbound traffic approaching the M6 Junction 8 northern merge from the M6 and M5 through to just south of M6 Junction 10A. They also provide timings at three intermediate locations between Junctions 8 and 10A including the merges at junctions 8 and 10.

2.1.3 Data Collected

The following data were collected:

- **ANPR Data** - Provides journey times between 6 locations on the route under study which are shown in Figure 2.1. This provides data for validating the JTV models.

- **MIDAS Counts** - Provides counts of traffic on each link and slip road in the study area which forms the basis for model development.

- **MIDAS Speeds** - Provides speed data on links which is used in model calibration and validation.

- **MTV Plots** - These provide a visual representation of MIDAS speed data. Using this data allows queue formation and propagation structures to be identified.

- **Incident Data** - This provides data on incidents and short term roadworks which may affect flow on the section of M6 under study.

- **Roadworks Data** - Provides data on longer term roadworks which may affect flow.

- **Events Data** - Provides data on events, e.g. sports events, which may have affected demand.

- **Wide Loads Data** - Provides data on wide loads which may affect flow.

- **Weather Data** - Provides data on weather conditions which may affect capacity.

Figure 2.1   Road Network Covered by the M6 Dataset, showing locations of ANPR Cameras

KEY
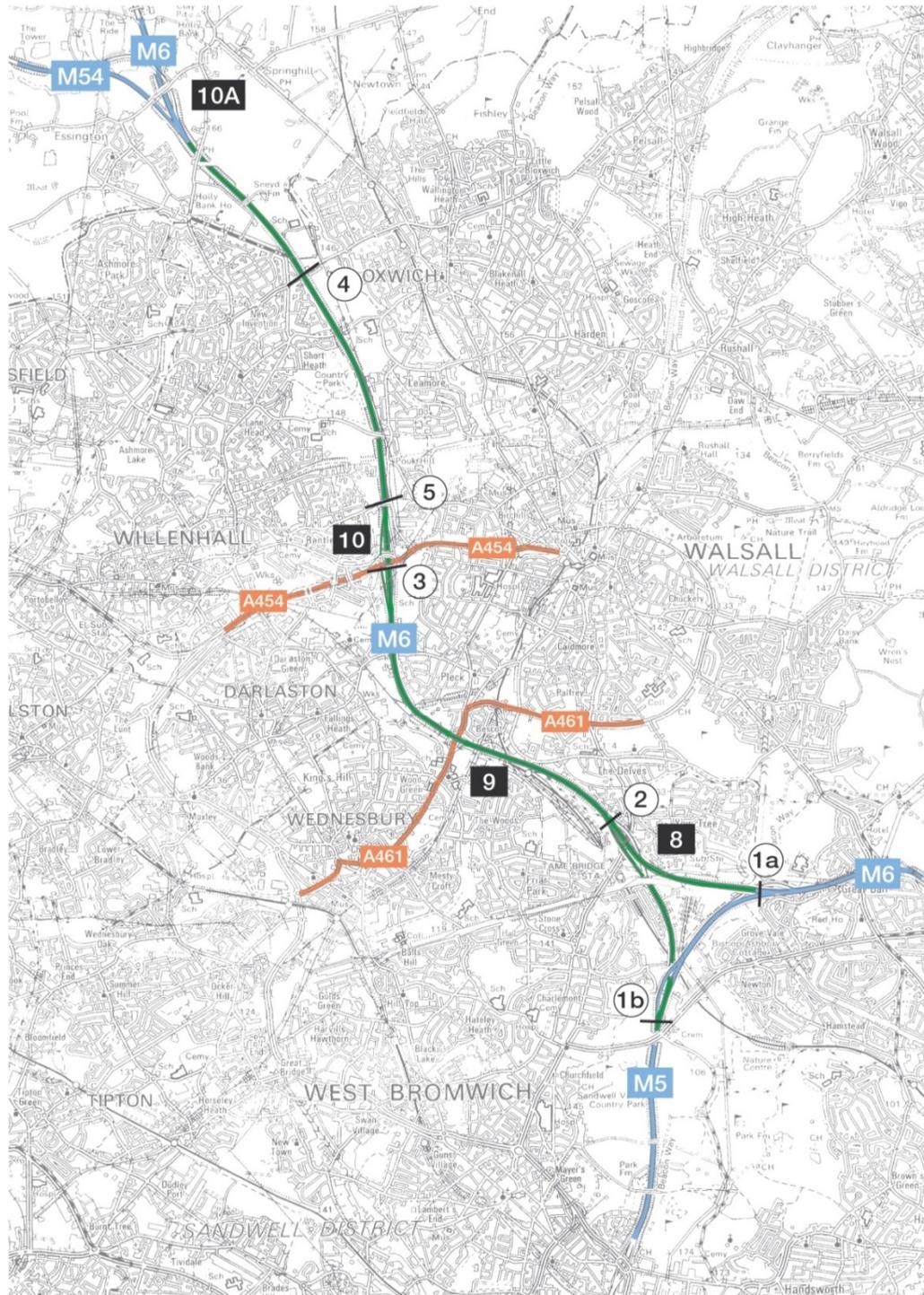
②   Route for data collection, and sites for ANPR cameras

M6 JUNCTIONS 8 TO 10A (NORTHBOUND)

2.2   Data Collection - Outcome

2.2.1   Overview

The M6 data collection was managed by TRL and is reported, with the exception of incident and wide load data collection, in 'Motorway Variability Data Collection in the West Midlands in 2003 - TRL PR/T/038/04, May 2004'. Incident data collection is reported in 'Updating and validating parameters for incident appraisal model INCA - TRL PR030, December 2005'.

Most of the data was collected for the period 30 December 2002 to 9 November 2003. The exceptions are the wide load data which was not collected for some of the main survey period, and the incident data which was not collected between 30 December 2002 and 5 January 2003.

2.2.2   ANPR Data

There were initial problems with recording ANPR camera clock corrections, but these were mainly resolved by mid-January 2003. ANPR data timings were corrected by TRL in accordance with the recorded clock corrections. ANPR observations were matched between cameras and outlier observations were removed using vehicle cohort analysis methods and manual inspection. The outlier observations are likely to be either vehicles which stopped or diverted within the route or else matching of repeat runs by the same vehicle. Comparison of MIDAS and ANPR data for a 4 week sample period for periods where both systems were operating gave an ANPR read rate averaged over 5 minute periods of 84% of vehicles with a minimum of 46% and a maximum of 92%.

2.2.3   MIDAS Count and Speed Data

Due to MIDAS system failures count and speed data was not available for the majority of MIDAS loops for much or all of a significant portion of 59 days during the survey period. The main failure periods were between 2/6/03 and 14/7/03 when the majority of loops were turned off and between 4/8/03 and 7/8/03 when the whole system failed. During the June/July failure there were no counts on the on-slips at Junctions 8, 9 and 10 or on the section of M6 between Junction 8 diverge and merge, while the count between Junctions 10 and 10A was intermittent. This combined with there being insufficient loops to plot queue propagation precludes useful data analysis for this period.

For shorter term loop failures TRL patched count data based on relevant observed periods. For the Junction 9 on-slip between 21/4/03 and 2/5/03 flows had to be estimated from counts at loops on the M6 upstream and downstream of the merge.

MIDAS speed data and useful MTV plots are also unavailable for the periods when MIDAS was not fully operational. The locations of MIDAS loops in the study area are shown in Figure 2.2.

2.2.4   Incident Data

After much of the incident data had been collected it was found that around 40% of the data comprised incidents which did not have any effect on the running carriageway, e.g. breakdowns on the hard shoulder. For the development of incident parameters for INCA only a sample of incidents were checked to determine whether they occurred on the carriageway. In view of this the incident data contains a high proportion of incidents which may have no significant effect on capacity. Initial analysis of the incident data showed that there are very few incidents prior to 27/1/03. This suggests that full incident data collection did not commence until 27/1/03.

Figure 2.2 MIDAS Detector and ANPR Camera Locations
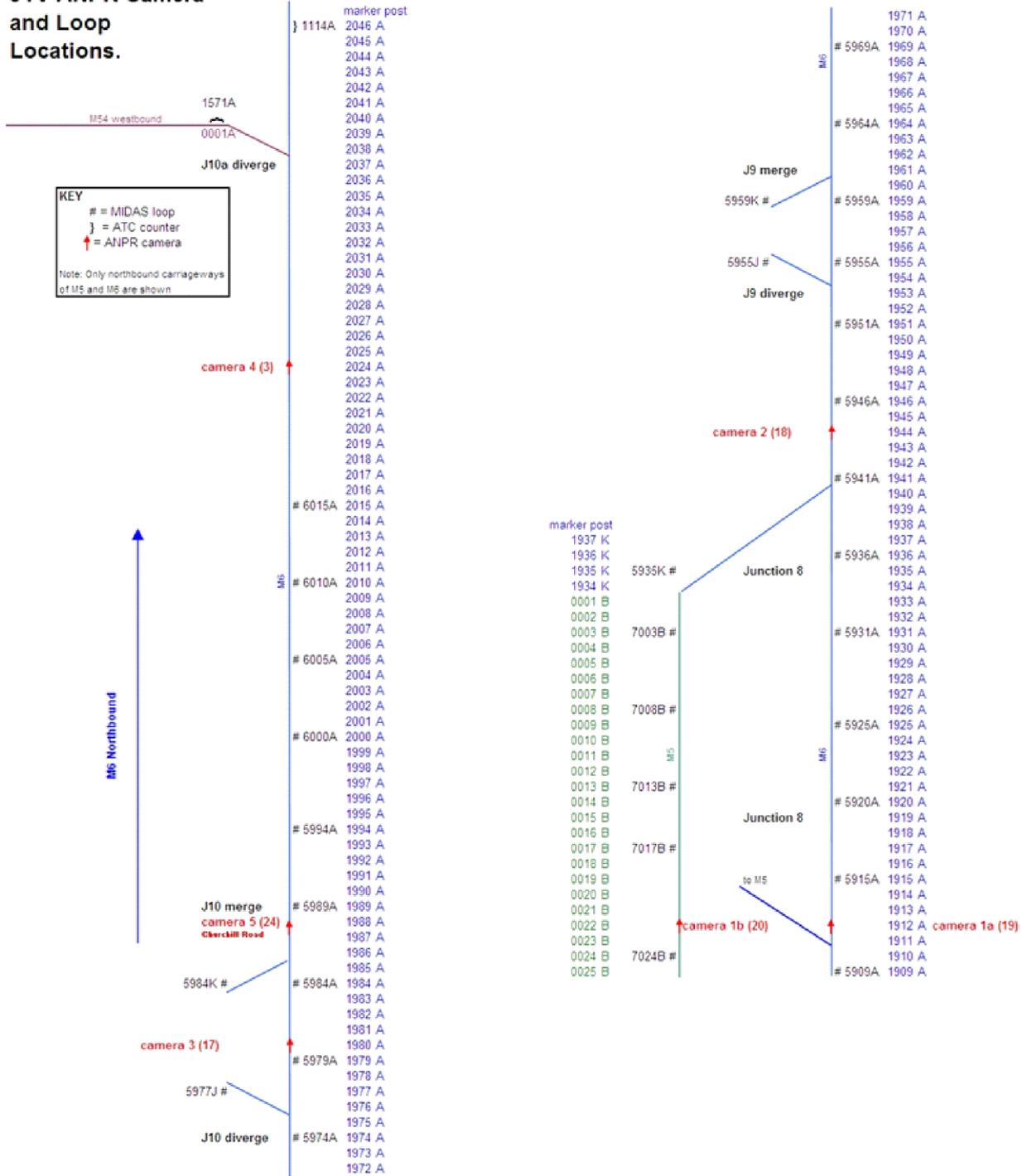
2.2.5   Other Data

The roadworks and events data were collected as specified.

Wide loads data collection was only carried out for 20 weeks. Analysis of the available wide loads data suggested that it is incomplete even for weeks during which data was collected. Also no evidence could be found on MTV plots that any of the wide loads had a noticeable effect on traffic flow on the motorways of interest. In view of this the wide load data has not been used in the analysis.

Weather data from the Hilton Park weather station close to the survey area is only available for weeks after 10/3/03. Prior to this, data is available from the Birmingham University weather station, but this data is more limited than at Hilton Park, and as the University is some distance from the survey area there are likely to be differences in weather conditions between the two locations. The system for recording visibility at Hilton Park was faulty and no visibility data is available from the University. It is understood that the onset of fog may have been recorded correctly, but that the fog indicator did not cancel when visibility improved.

2.2.6   Data Availability

Due to the lack of incident data and initial problems with the MIDAS and ANPR data the data prior to 27/01/2003 was not suitable for further analysis. The data availability for the remaining survey weeks is summarised in Table 2.1. Because of the interest in periods of excess demand we have generally concentrated on weekday rather than weekend data.

Allowing for days when there were major MIDAS failures this left 91 weekdays in school terms and 54 in school holiday periods excluding bank holidays.

Table 2.1 M6 Survey Summary

Week No (TRL No)	Week Starting	School Holiday	MIDAS Days Off	Incident Data	Weather Data	Neutral Weekdays	Holiday Weekdays
5	27/01/2003		2	1	BU	3	
6	03/02/2003		2	1	BU	3	
7	10/02/2003		1	1	BU	3	
8	17/02/2003	1		1	BU		5
9	24/02/2003	1		1	BU		5
10	03/03/2003			1	BU	4	
11	10/03/2003			1	HP	5	
12	17/03/2003			1	HP	5	
13	24/03/2003			1	HP	5	
14	31/03/2003			1	HP	5	
15	07/04/2003			1	HP	4	
16	14/04/2003	1		1	HP		5
17	21/04/2003	1		1	HP		5
18	28/04/2003			1	HP	3	
19	05/05/2003			1	HP	3	
20	12/05/2003			1	HP	5	
21	19/05/2003			1	HP	4	
22	26/05/2003	1		1	HP		5
23	02/06/2003		7	1	HP		
24	09/06/2003		7	1	HP		
25	16/06/2003		7	1	HP		
26	23/06/2003		7	1	HP		
27	30/06/2003		7	1	HP		
28	07/07/2003		7	1	HP		
29	14/07/2003		2	1	HP	3	
30	21/07/2003			1	HP	4	
31	28/07/2003	1		1	HP		5
32	04/08/2003	1	5	1	HP		
33	11/08/2003	1		1	HP		5
34	18/08/2003	1		1	HP		5
35	25/08/2003	1		1	HP		4
36	01/09/2003	1		1	HP	2	
37	08/09/2003			1	HP	5	
38	15/09/2003		1	1	HP	4	
39	22/09/2003			1	HP	5	
40	29/09/2003		1	1	HP	4	
41	06/10/2003			1	HP	5	
42	13/10/2003			1	HP	4	
43	20/10/2003	1		1	HP		5
44	27/10/2003	1	1	1	HP		5
45	03/11/2003		1	1	HP	3	
Total			**58**			**91**	**54**

Weather Data Sources:
BU - Birmingham University; HP - Hilton Park

3 DATA PROCESSING AND ANALYSIS

3.1 ANPR Data

The individual vehicle matched registration number data was cross tabulated to give journey times for each inter-camera movement for each 5 minute period of the survey. These journey times excluded observations which had been flagged as being in error. The inter camera movements for which data was matched and cross tabulated are as follows:

- 1a to 2
- 1a to 4
- 1b to 2
- 1b to 4
- 2 to 3
- 2 to 5
- 3 to 4
- 5 to 4

The next stage was to identify periods which are affected by incidents or roadworks. This was based on the analysis of journey times for the route between cameras 1a and 4 which covers all of the other routes on the M6.

All the incident data, which covers motorways for the whole of the West Midlands Region, was processed to identify the link on which the incidents occurred. Incidents which might affect journey times between cameras 1a and 4 were then selected. These comprised incidents occurring between M6 Junctions 7 and 16 and between M54 Junctions 0 and 7. Motorway sections downstream of M6 Junction 10A were included since blockbacks were known to occur.

The roadworks data was analysed in the same way as the incident data to identify those roadworks which might affect journey times in the study area. Most of the roadworks data covered multiple days and was not specific about on which days the roadworks occurred. In consequence when roadworks periods were flagged a very high proportion of ANPR periods were excluded as being affected. In view of this it was decided not to flag the non-specific roadworks periods.

The weather data for both Hilton Park and Birmingham University was summarised and appended to the ANPR databases.

It is known that many of the reported incidents and roadworks had no effect on capacity as they caused no significant lane blocking. To identify periods where incidents had a significant effect on journey times, the data was divided into two sets as follows:

- days during school term times; and
- days during school holidays.

Initially this was done on a week basis though Bank Holidays were included in the school holidays. However, analysis of journey times for individual days showed that days adjacent to Bank Holidays had significantly higher journey times than the average for the same day of the week. This is likely to be due to holiday traffic travelling on the day before and after the Bank Holiday weekends. In view of this these days were also moved from the term time dataset to the holiday dataset.

Days for the period for which little MIDAS data is available (2/6/03 to 13/7/03) were excluded from the analysis, since there would be no usable MTV plot data to facilitate incident identification and in any case the count data was not available for modelling.

Initially, to identify incident- and roadworks-affected periods (hereafter referred to as incidents) journey times during the duration of those incidents which might have an effect and afterwards were compared with the journey times for the same 5 minute periods on the same day of the week for unaffected days in the dataset (school term or school holidays). Any period where journey time was outside 2 standard deviations (the 95% confidence interval assuming a normal distribution) of the mean for non-incident affected periods was flagged as being affected by the incident.

All incident affected periods were then manually checked against the incident and roadworks files with the support of MTV plots. This confirmed that the effect was caused by an incident rather than just by flow breakdown due to excess demand.

Analysis of journey time graphs showed that there were other possible incidents which had not been included in the incident or roadworks datasets. These are probably either roadworks where the timing was vague or incidents off the motorway network which obstructed off-slips. Such occurrences were identified using MTV plots and these were treated in the same way as other incidents.

On the basis of the above analysis, the incident effect flags were set in the incident databases to identify 5 minute periods affected by incidents. This allowed those days which have no incident effect for any period to be identified as potential candidates for model calibration excluding incidents.

A summary of the incident effect analysis by Term Time day is set out in Table 3.1. Based on the incident rates derived by TRL for INCA from the West Midlands incident data (TRL, 2005), the daily incident rate northbound between M6 Junctions 8 and 10A would be expected to be around 0.4. Overall in Term Time incidents occur on 42% of days which is consistent with the TRL incident rates. There are 139 days with journey time data of which 80 had no significant incidents. The table gives the mean maximum journey times for each day between ANPR cameras 1a and 4. On average, incidents increase the mean maximum journey time by 40%.

Table 3.1 Summary of Days - Term Time

Day	Number of Days		Mean Maximum JT Cameras 1a to 4 (mins)	
	All Days	Incident Free Days	All Days	Incident Free Days
Monday	21	10	13.000	9.736
Tuesday	21	15	18.926	15.483
Wednesday	22	15	23.761	21.290
Thursday	20	8	33.042	20.858
Friday	19	10	28.049	26.090
Saturday	18	11	22.320	7.241
Sunday	18	11	15.789	9.422
All Days	**139**	**80**	**22.108**	**15.750**
Weekdays	**103**	**58**		

A summary of the incident effect analysis by School Holiday day is set out in Table 3.2. Overall significant incidents occur on 37% of School Holiday days, which again is consistent with the TRL incident rates. There are 106 School Holiday days of which 67 are not affected by significant incidents. For School Holiday days incidents increase the mean maximum journey time by 28%.

Table 3.2 Summary of Days – School Holidays

Day	Number of Days		Mean Maximum JT Cameras 1a to 4 (mins)	
	All Days	Incident Free Days	All Days	Incident Free Days
Monday	14	6	17.977	12.944
Tuesday	14	8	19.660	17.017
Wednesday	13	7	24.213	19.438
Thursday	15	12	27.775	22.899
Friday	16	10	30.494	24.445
Saturday	17	14	17.785	14.710
Sunday	17	10	18.964	9.786
All Days	**106**	**67**	**22.367**	**17.506**
Weekdays	**72**	**43**		

The above comparison with the TRL incident rates validates the incident effect analysis.

3.2 MIDAS Counts

For periods where the MIDAS system showed an error, counts had been patched by TRL. However, analysis of the data showed that there were a significant number of periods with a zero count in one or more lanes, where the loop was classified as working, but had in fact been in error. In view of this it was necessary to analyse all zero counts between 0600 and 2200 (night time counts were not analysed due to the high expected incidence of genuine zero counts) to identify whether each zero was genuine or an error.

For sites on the M6 main line and the M5 slip at Junction 8 (J8) it was possible to identify genuine zeros by analysis of MTV plots. Genuine zeros occur either where flows are very low, typically at night or on Sunday or Bank Holiday early mornings, or when flow is disrupted by an incident.

For slip roads, where there are no MTV plots, it was necessary to compare the slip road counts with the difference between the adjacent M6 main line counts. This highlighted the most significant occurrences of "observed" zero periods which are actually in error.

Periods with valid zeros were flagged as Good. Periods with error zeros were flagged as not Good and a new flag Bad no Patch was set to denote that these had not been patched. This facilitates the exclusion of periods with error zeros.

Analysis of the slip road data showed two long term problems, relating to J9m and J10d.

TRL had noted that the J9m loop was not working correctly between 21/4/03 and 2/5/03. They stated that data for this period was patched but no evidence of this could be found. Analysis of data for this loop suggested the following:

- from the beginning of the survey until 25/3/03 the lane loop counts appeared to be reversed i.e. the lane 1 count is recorded as lane 2 and visa versa;
- between 25/3/03 and 2/5/03 the loop on lane 1 was not working at all; and
- between 2/5/03 and 8/5/03 the lane 1 loop was only working intermittently.

To resolve this problem it was necessary to patch the J9m data between 25/3/03 and 8/5/03 using a methodology similar to that proposed by TRL. This estimates the J9m counts based on the difference between the counts on the adjacent M6 main lane sites. Length group and lane counts were patched based on average one minute distributions for the period when the loop was working.

A similar problem was found for J10d between 10/2/03 and 16/2/03. This data was patched using the same methodology as for J9m.

3.3 MIDAS Speeds

Analysis of the MIDAS speed data showed a significant number of periods where the speed was set to the 255 error code. This mainly occurs when the primary loop on a link was faulty and an alternative loop was used for counting. To minimise this problem for M6 main line links, speeds were extracted for each time period for the loop which was used for the count for that period. However, this did not totally solve the problem.

Where possible the remaining 255 speeds were patched using the speed from the alternative loop on the link. Where this could not be done the loop speed was set to blank and flagged with a Bad Speed Flag.

3.4 Demand Data

For the Demand Model development 5 minute count demand data was produced for each of the 4 entries and 4 exits from the M6 junction 8 merge to junction 10A northbound. The data has been selected from the MIDAS count database only for periods between 0600 and 2200 where there is no incident effect on travel times between camera 1a and 4 and where the count data for every minute in the period is flagged as good.

3.5 Final Databases

The finalised databases output from the processing are described in detail in 'User Guide to Processed M6 Data – May 2010'.

Due to deficiencies in data in one or more datasets it is not possible to produce comprehensive data for all days. While in principle 41 weeks were surveyed, which should have yielded 205 weekdays (though some allowance needs to be made for Bank Holidays), the actual usable set is far lower, and this was largely for reasons outside the surveyors' control (in particular, equipment failure). The requirement for incident-free and fully observed data imposes severe restrictions on the final data set.

For the S-Paramics model development, days with no incident effects and full demand data between 0600 and 2200 were required. For this purpose 25 weekdays were identified with no incident effects and only 10 minutes or less of missing MIDAS count data. For the demand and aggregate modelling, where some of the missing demand data could be interpolated, 67 weekdays with no incident effects but which had flow breakdown at one or more junction merges were identified.

## 3.6 ANPR Journey Time Analysis

3.6.1 Overview

To illustrate the journey time and TTV profiles for M6 Junction 8 to 10A the analysis of ANPR data for term time days is discussed below.

The distance between cameras 1a and 4 is about 11.18 Km, so travelling at 70 mph (113 kph) the distance would be covered in just under 6 minutes. For term time weekdays the journey time averaged for individual days of the week, for 5 minute periods between 0600 and 2200 which are not affected by incidents, is 8.4 minutes which is equivalent to a speed of 80 kph. The minimum journey time is 6.2 minutes (108 kph) for 2145 on a Friday probably due to the low proportion of heavy goods vehicles on a Friday evening. The maximum is 25.3 minutes (27 kph) for 1640 on a Friday mainly due to queueing caused by flow breakdown at the junction merges

3.6.2 Journey Time Profiles

Average Term Time journey time profiles between cameras 1a and 4 are set out in Figure 3.1. Each point in the figure is the average journey time in a 5 minute period, over all relevant days, **without** excluding periods affected by incidents. This shows that on all days except Saturday and Sunday the maximum journey time occurs at around 1700. For Sunday it occurs around

1400 and for Saturday around 1230. The volatile Saturday journey times around 1230 are caused by a single very serious incident where in some 5 minute periods no vehicles were able to complete this journey and in adjacent periods journey times were very high, reaching a maximum of 198 minutes.

Fig 3.1 Average Journey Times by Term Time Day 1a to 4 - Including Incidents
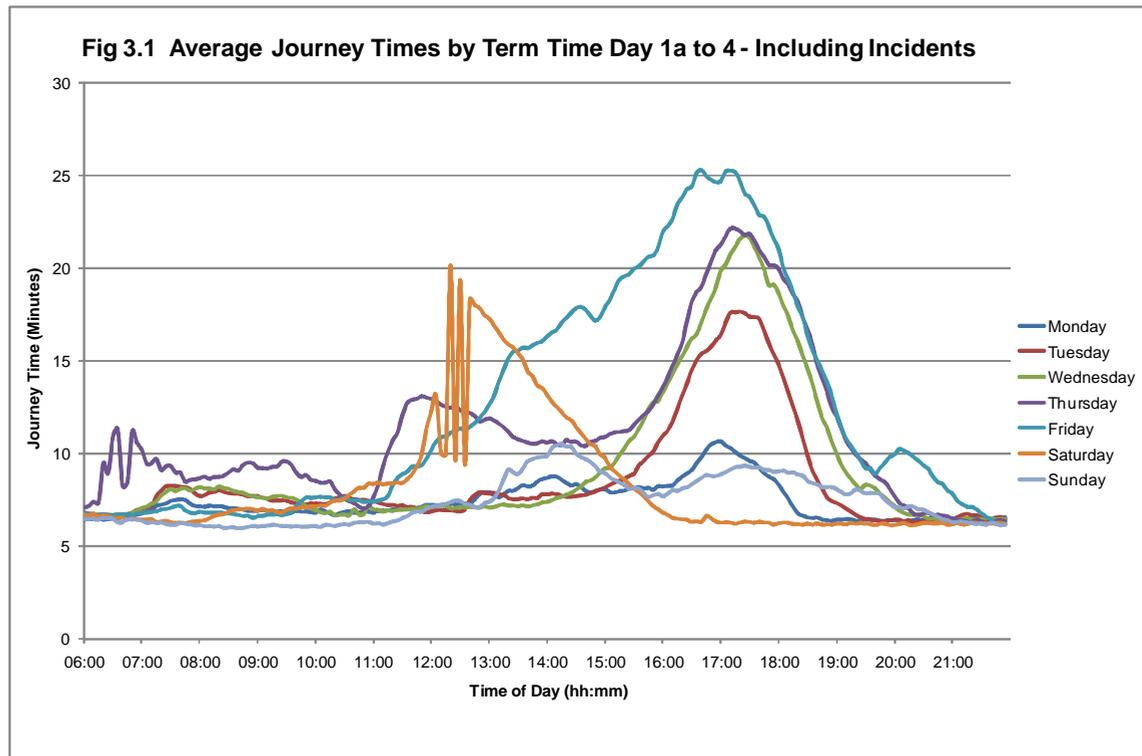
Average Term Time journey time profiles by day excluding those 5 minute periods affected by incidents are set out in Figure 3.2. With incident affected periods excluded all days except Saturday follow a similar pattern with the highest journey times being around 1700. On Saturdays the highest journey times are around 1100. The peaks are most pronounced on Tuesdays to Fridays, while the profiles for Saturday to Monday are much flatter.

Fig 3.2 Average Journey Times by Term Time Day 1a to 4 - Excluding Incidents
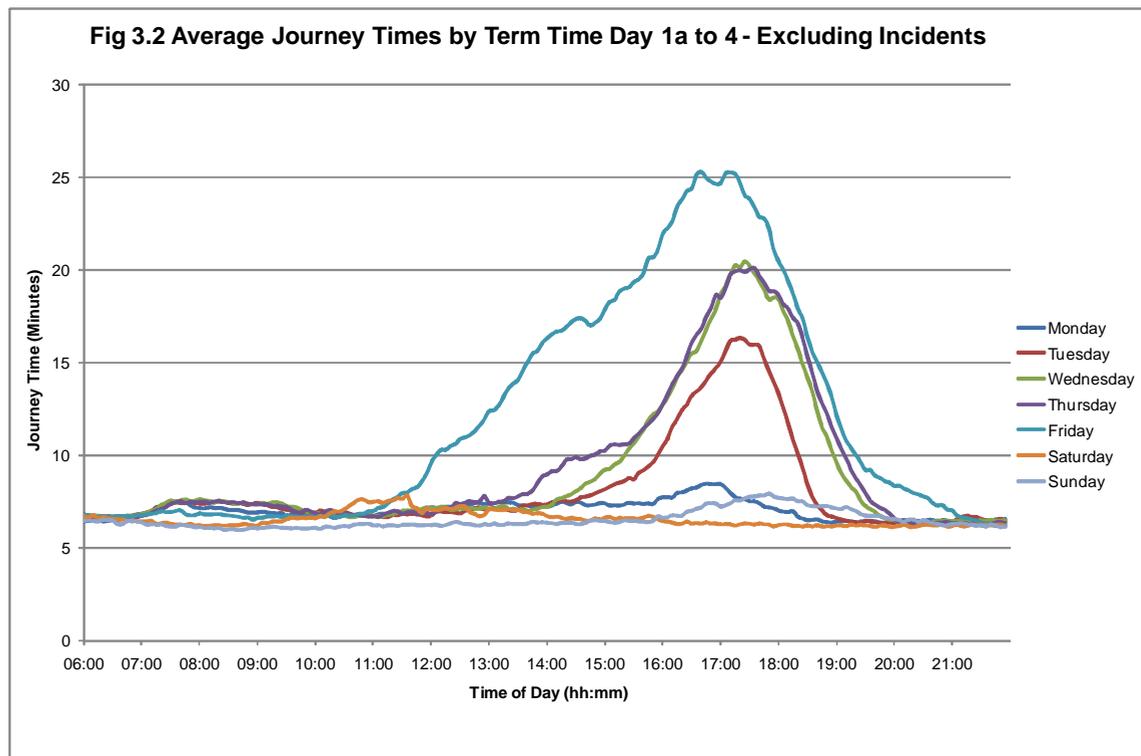
On all days prior to around 1200 the journey times are relatively constant, marginally higher than the 6 minutes which would apply to an average speed of 70 mph. This shows that the higher journey times in the mornings are mainly caused by incidents rather than flow breakdown due to excess demand.

Fridays on average have by far the highest and most sustained increased journey times, with the increased journey times occurring between 1100 and 2100, reaching a peak of 25 minutes around 1700, implying an average speed of about 17 mph which must be the product of stop-go driving (queueing). For Wednesday and Thursday the increased journey time profiles are similar reaching a peak of 20 minutes around 1700 and falling away by 2000. However, for Thursdays the increased journey times start around 1330, while on Wednesdays they start later at around 1430. On Tuesdays the increased journey times are between about 1430 and 1900 peaking at 17 minutes at around 1700. On Mondays and Sundays there is little increase in afternoon journey times while on Saturdays there is none.

3.6.3   TTV Profiles

Coefficients of Variation (CV – the standard deviation to mean ratio) of Term Time journey times including periods affected by incidents are set out in Figure 3.3. This shows that CVs are very volatile, reaching as high as 2.2 for Saturday and 2.0 for Thursday. Again the very volatile CVs between 1200 and 1300 on Saturday are caused by the very serious incident where vehicles failed to complete the route in some 5 minute periods.

Fig 3.3 CV of Journey Times by Term Time Day 1a to 4 - Including Incidents
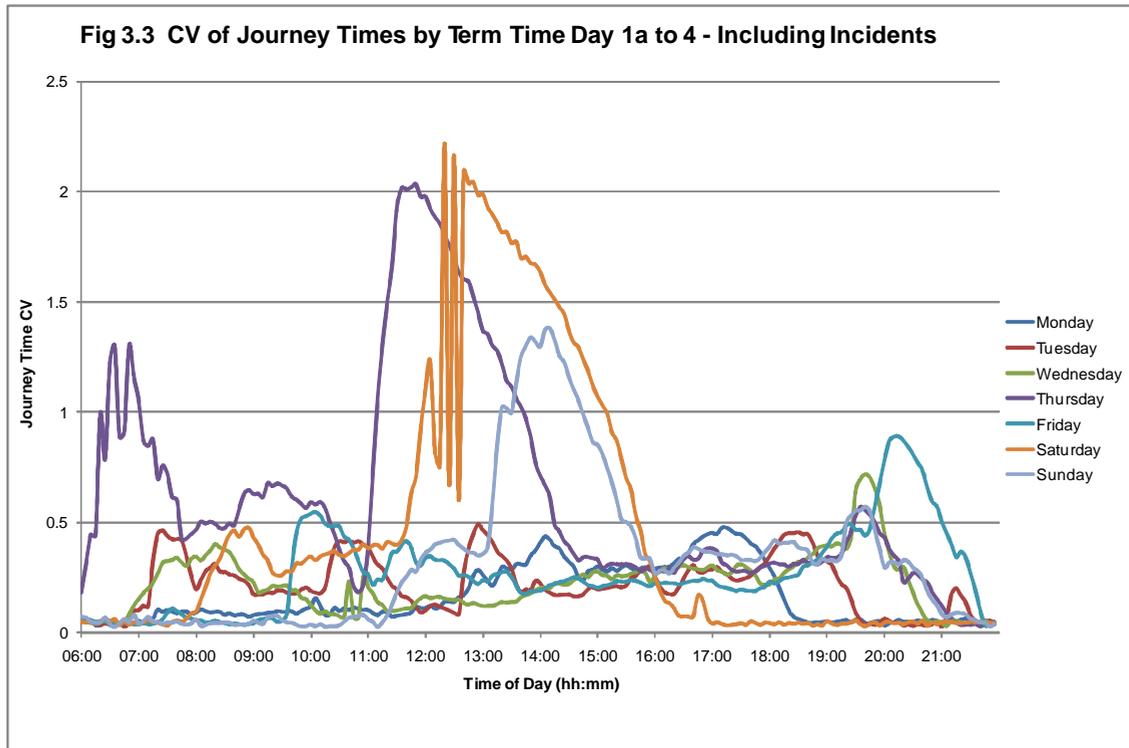

CVs of Term Time journey times excluding incident affected periods are set out in Figure 3.4: note that the scale on the vertical axis has changed. The highest CV is 0.5 for Friday. This demonstrates that the very high CVs noted above are caused by incidents. With the exception of Saturdays the CVs are always lower in the morning (generally not greater than 0.1), and higher in the afternoon (generally between 0.2 and 0.4 for most weekdays). For weekdays the highest CVs tend to occur in the late afternoon around the time that average journey times decline from their peak.

Fig 3.4 CV of Journey Times by Term Time Day 1a to 4 - Excluding Incidents
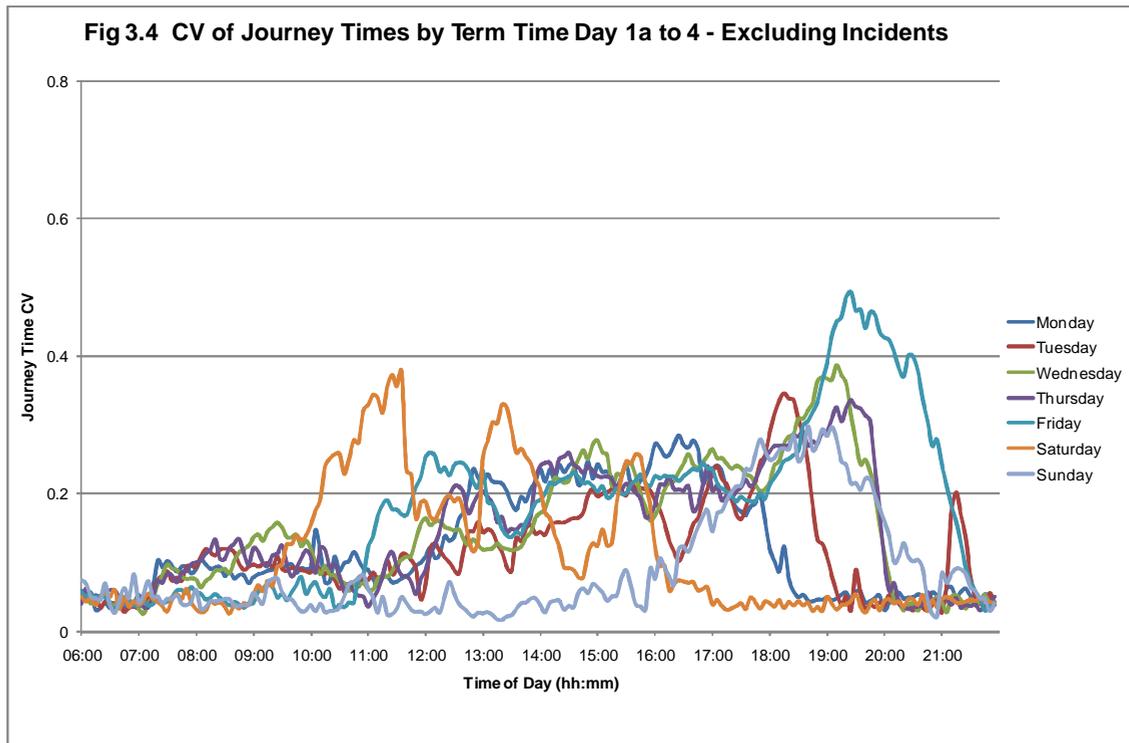
In order to illustrate the day-to-day variation, Journey time profiles for each individual term time Monday are shown in Figure 3.5. Generally the journey times for individual days are well grouped and less than 10 minutes. However, there is one obvious outlier, 22/9/03, when journey times were high between 1200 and 1700 peaking at over 16 minutes at around 1630. This may be due to a rainstorm with 18.5mm of rain between 1150 and 1450, which was intense at the beginning with the equivalent of 25.2mm per hour in the 5 minute period at 1210. This period had the highest recorded rainfall of any 5 minute period for the whole of the period surveyed by the Hilton Park weather station.

**Fig 3.5 Monday Term Time Journey Times 1a to 4 - Excluding Incidents**
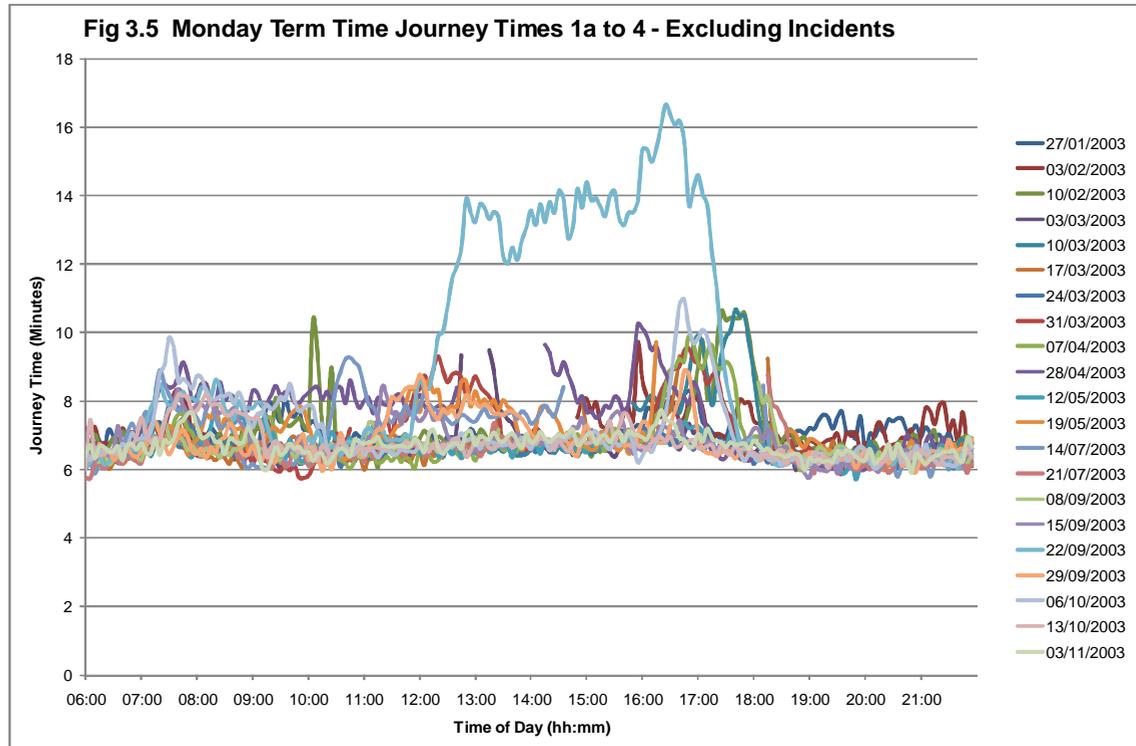
Tuesday individual day travel time profiles are set out in Figure 3.6. In general the individual day travel time profiles are well grouped. However an obvious outlier is 28/1/03 which had much higher than average peak journey times. On this day flow breakdown occurred earlier than usual at the J10 merge due to above average demand on the on-slip. Due to blocking back this would have reduced capacity on the outflow from the J8 merge earlier than on other days thus causing higher delays. The earlier flow breakdown at J10m may have been due to additional demand at J10m possibly caused by traffic diversion to avoid earlier roadworks on the M5.

Fig 3.6 Tuesday Term Time Journey Times 1a to 4 - Excluding Incidents
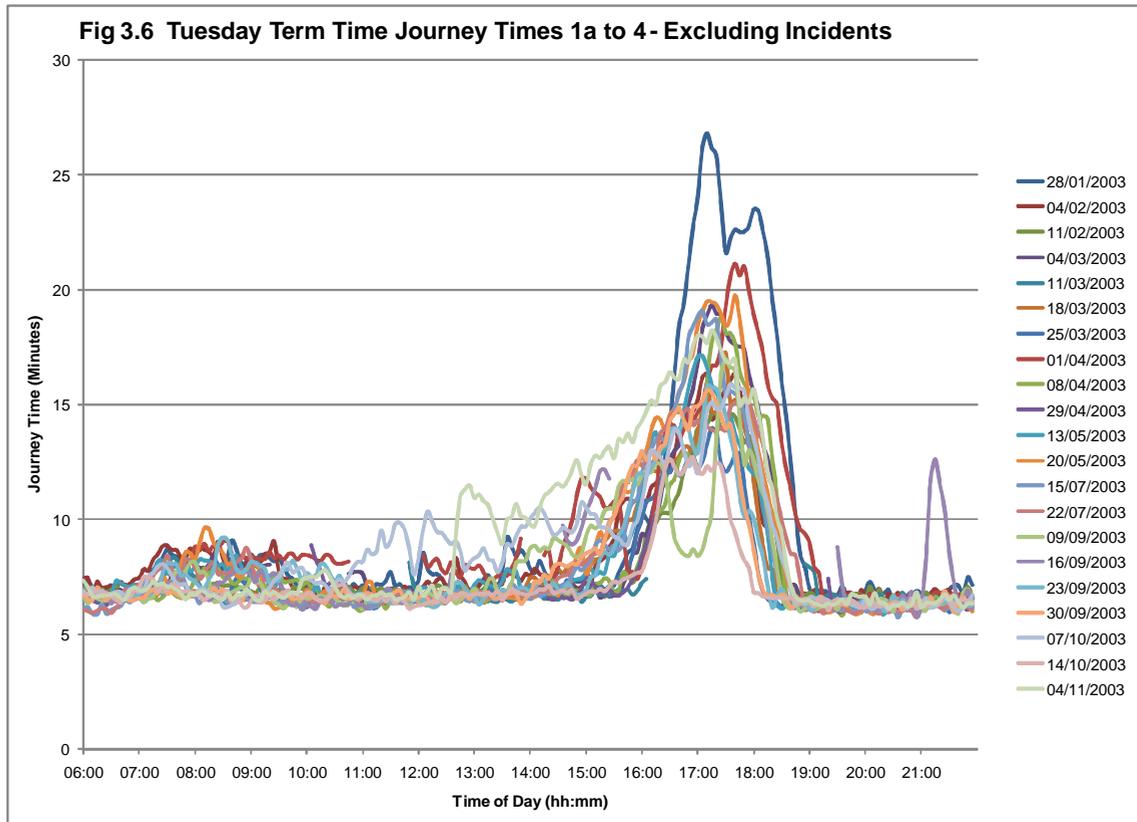
Individual day travel time profiles for Wednesday are shown in Figure 3.7. These profiles are generally well grouped.

Fig 3.7 Wednesday Term Time Journey Times 1a to 4 - Excluding Incidents
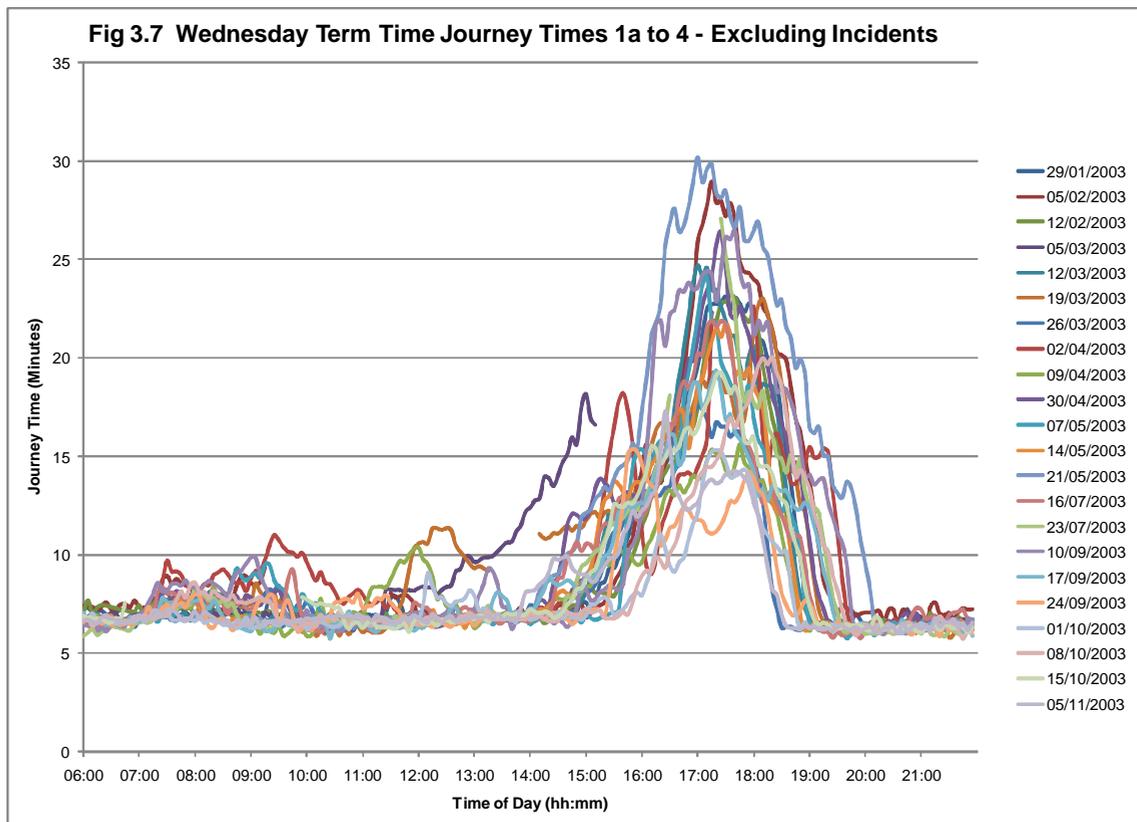
Thursday individual day travel time profiles are set out in Figure 3.8. In general these profiles are well grouped.

Fig 3.8 Thursday Term Time Journey Times 1a to 4 - Excluding Incidents
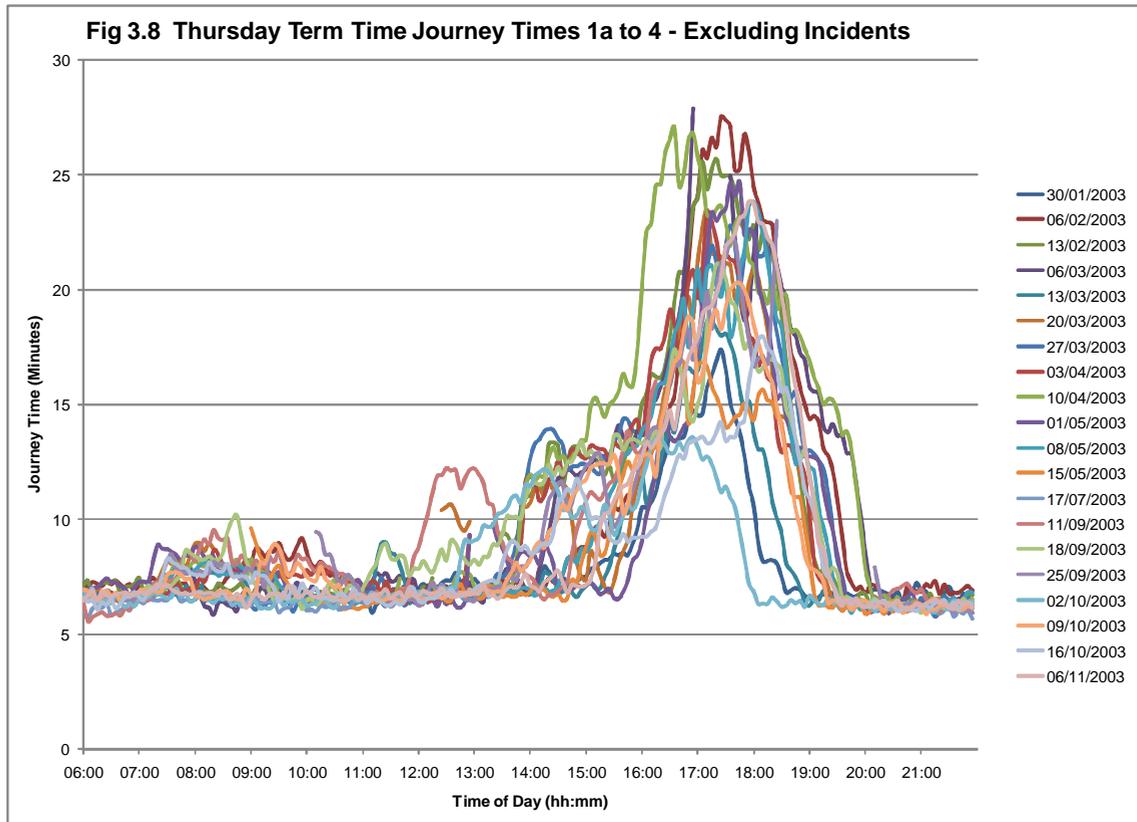
Individual day travel time profiles for Friday are set out in Figure 3.9. These are generally well grouped with the exception of 21/3/03 where travel times are relatively low between 1400 and 1730. Analysis of the demand at the J8 merge compared with adjacent Fridays gave no obvious explanation for this difference, although it might relate to very detailed differences in the demand profile which allowed flow to recover from a breakdown state around 1400.

Fig 3.9 Friday Term Time Journey Times 1a to 4 - Excluding Incidents
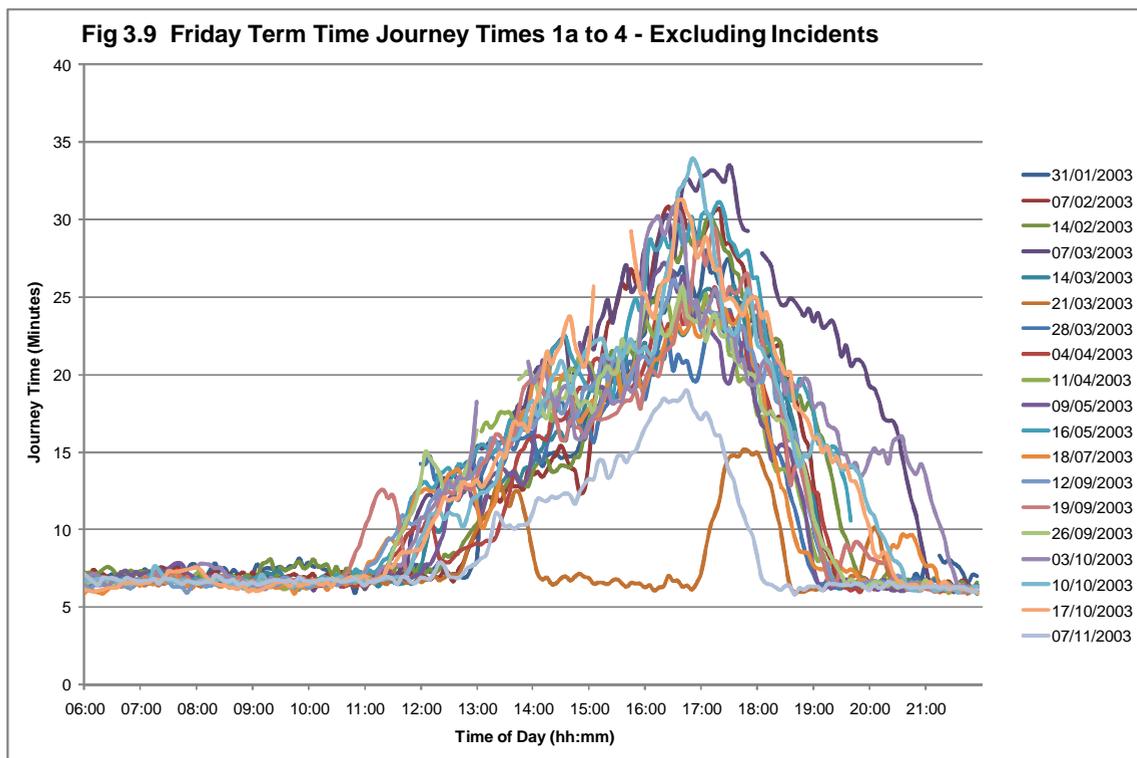
The analysis of journey time profiles for individual days confirms that flow breakdown due to excess demand occurs on all term time Tuesday, Wednesday, Thursday and Friday afternoons. For each day of the week the temporal distribution of the excess journey times caused by excess demand is similar for all occurrences of that day. This suggests that the cause of the excess journey times is systematic and is therefore amenable to modelling.

4 FLOW BREAKDOWN

4.1 Introduction

This Section discusses flow breakdown in the particular context of M6 Junction 8 to 10A, describing the phenomenon as illustrated in MTV plots. It draws on research undertaken by TRL for the M25, which is reviewed and extended, and sets out the implications for the modelling of flow breakdown.

4.2 TRL Research

The MTV system of illustrating traffic flow patterns on motorways using MIDAS speed data was developed by TRL. Their understanding of flow breakdown mechanics as illustrated by MTV plots for the M25 is set out in 'Insight Report INS003 - Speed, flow and density of motorway traffic' (TRL, 2009). The objectives of this report were:

- to provide a broad overview of existing theories of traffic flow; and
- to examine the results and implications of these theories in the context of speed and flow data available from HATRIS.

The report concentrates on motorways for which comprehensive data is available from the MIDAS system.

Based on speed flow relationships, (TRL, 2009) categorises two traffic flow states as follows:

- free-flowing with a relatively small decline in speed as flow increases, which is the state addressed by speed flow curves in assignment models; and
- congested flow where flow declines as speed declines.

The transition from free flow to congested flow occurs when flow reaches capacity, at which point flow breakdown occurs to cause the transition into the congested flow state. Recovery from the congested state to the free flow state occurs when flow reduces to a level which allows this recovery.

Flow breakdown mechanics are illustrated by analysis of MTV plots for M25 Junction 10 to 12 clockwise (A3 to M3). These show that flow breakdown occurs generally at specific 'seed points' which are believed to be locations within links where effective capacity reduces, e.g. locations with gradients. Once flow breakdown has occurred it propagates upstream in 'shockwaves'. In the case of M25 Junction 10 to 12 the main 'seed point' is around 1.5 km downstream of the Junction 11 merge. The shockwaves propagate back to Junction 10 diverge and probably beyond (the MTV plots do not extend any further).

The M25 Junctions 13 to 16 clockwise section (A308 to M40) shows a similar picture, with a 'seed point' about 2.5 km downstream of the Junction 15 merge. 'Shockwaves' propagate upstream beyond Junction 13. In the case of both M25 sections, speed and flow recover quickly at the 'seed point'. This gives a series of 'shockwave' flow breakdowns for a short period, between which are longer flow recovery periods.

(TRL, 2009) also shows the effect of an incident which suddenly reduces capacity by blocking one or more lanes. This causes a queue to build up behind the incident location, which only begins to clear when the demand joining the back of the queue falls below the queue outflow capacity, either because the obstruction caused by the incident is cleared or demand falls. The queue for the example incident downstream of Junction 15 at its longest point propagates back beyond Junction 14. This gives rise to a roughly triangular area of flow breakdown on the MTV plot.

TRL note that 'shockwaves' propagate upstream at around 19 km/h.

Two types of methodology for modelling flow breakdown are discussed:

- speed-density models; and
- models based on queueing theory.

4.3 Comments on the TRL Research

The concepts of 'seed points' and 'shockwaves' are useful in understanding flow breakdown. Given the intermittent nature of the shockwaves they are probably caused by short spikes in demand which transform flow from 'free flow' to 'congested flow'. The slowing of vehicles then propagates upstream as vehicles travelling behind the vehicles which first experience flow breakdown are forced also to reduce speed. This constrains flow, as the capacity at the head of the shockwave is effectively reduced by the instability of flow. This in turn reduces demand downstream to allow the 'free flow' state to be recovered. Thus the 'shockwave' propagates upstream, but as it moves it allows vehicles downstream of its head to regain free flow speed. This is clearly a condition which should be modelled using the speed-density type of approach.

In the case of M25 Junction 10 to 12 the 'shockwave' appears to be the main type of flow breakdown. This regime may be related to lane drops at junctions which reduce increases in demand relative to capacity at merges.

However, in the case of the Junction 13 to 16 section there is clearly a different phenomenon occurring at the Junction 15 (M4) merge, where a high volume of traffic merges. At this point the flow breakdown is relatively constant for a long period and it propagates upstream of the flow breakdown point in a similar manner to that from an incident, except that the head of the queue stays at the same location. It seems appropriate to term this a 'standing wave'. As with incidents, it seems most appropriate to model 'standing waves' using a queueing theory approach.

There is clearly interaction between the 'shockwaves' and the 'standing waves'. It is notable that the 'standing wave' at Junction 15 merge begins when a 'shockwave' from downstream reaches it. It is also clear that later 'shockwaves' propagate through the 'standing wave'.

The question arises as to why flow does not regain the 'free flow' regime at the 'standing wave' location after the 'shockwave' has passed upstream. The answer is that the margin of flow below capacity given by the shielding effect of the shockwave moving upstream is taken up by demand which has been stored on the on-slip.

4.4 M6 Junction 8 to Junction 10A

M6 Junction 8 to 10A has two important features which do not occur on the section of M25 analysed by TRL:

- there is no lane drop at any of the junctions; and
- the M5/M6 Junction at Junction 8 is a T-junction, hence the merging flows are likely to be relatively much higher than those at M25 junctions where motorways intersect.

To demonstrate what is happening on this section the MTV plots for Friday 10 October (this is a typical Friday which is the day of the week which has the most severe flow breakdown) are set out in Figures 4.1 to 4.3. Note that these plots were produced by TRL software, and it is not possible to edit them. It is unfortunate that they appear to be dominated by the blue section relating to upstream loops which were not operational. It is only the upper third of the diagram which is relevant to this study.

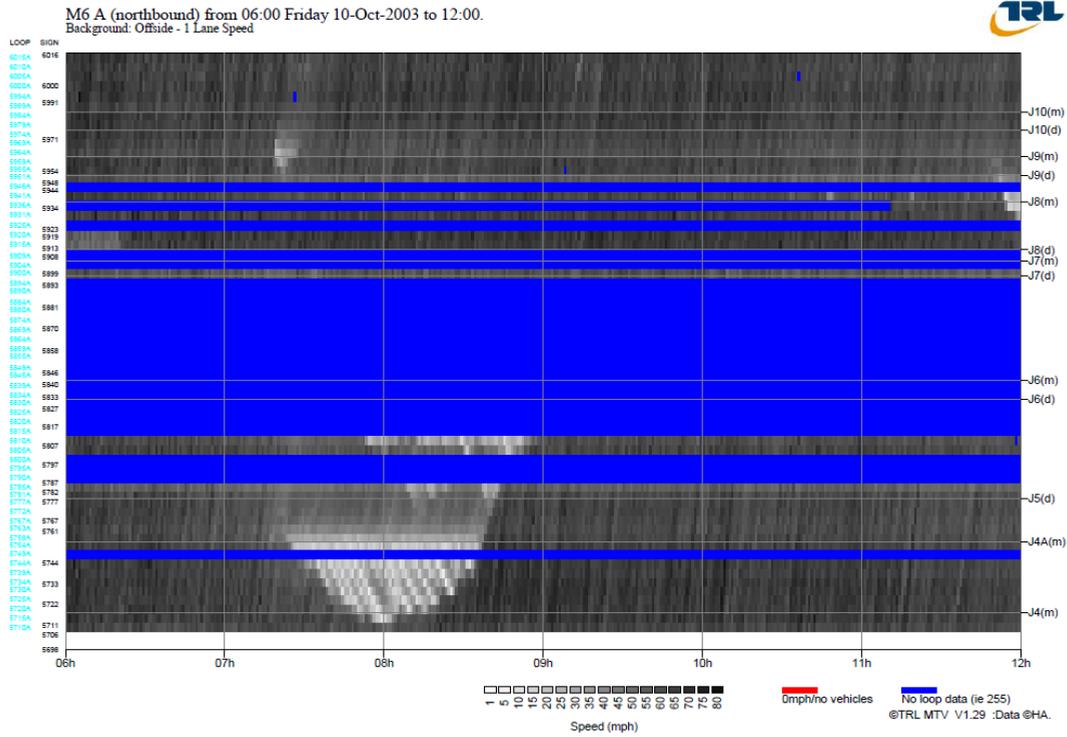**Figure 4.1 M6 Northbound MTV Plot 10/10/03 0600-1200**

M6 A (northbound) from 06:00 Friday 10-Oct-2003 to 12:00.
Background: Offside - 1 Lane Speed

Figure 4.2 M6 Northbound MTV Plot 10/10/03 1200-1800

M6 A (northbound) from 12:00 Friday 10-Oct-2003 to 18:00.
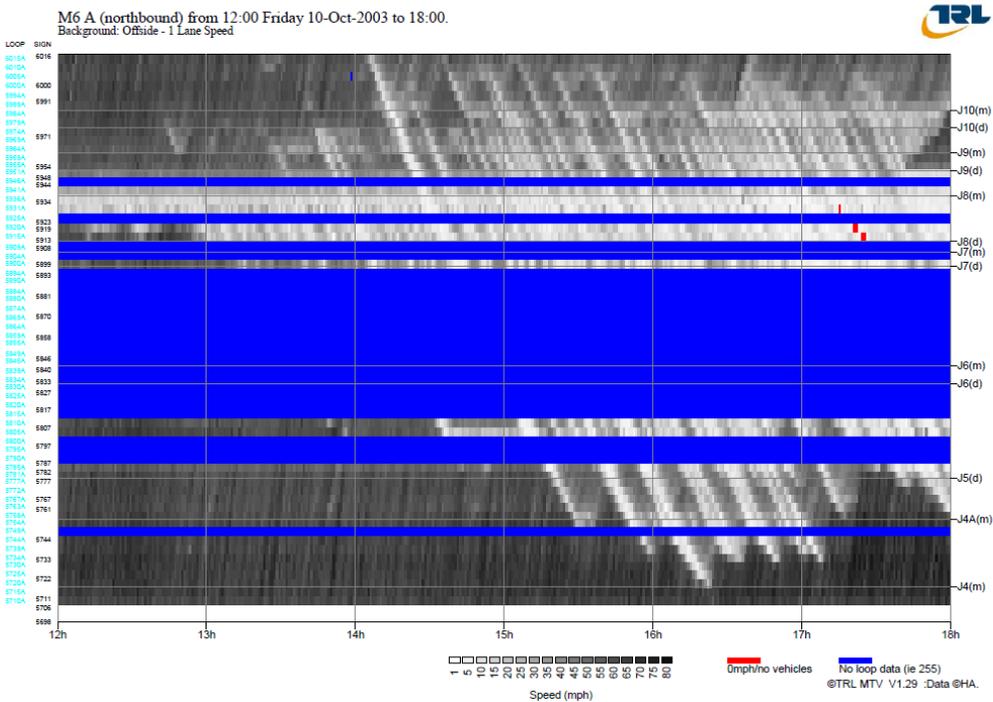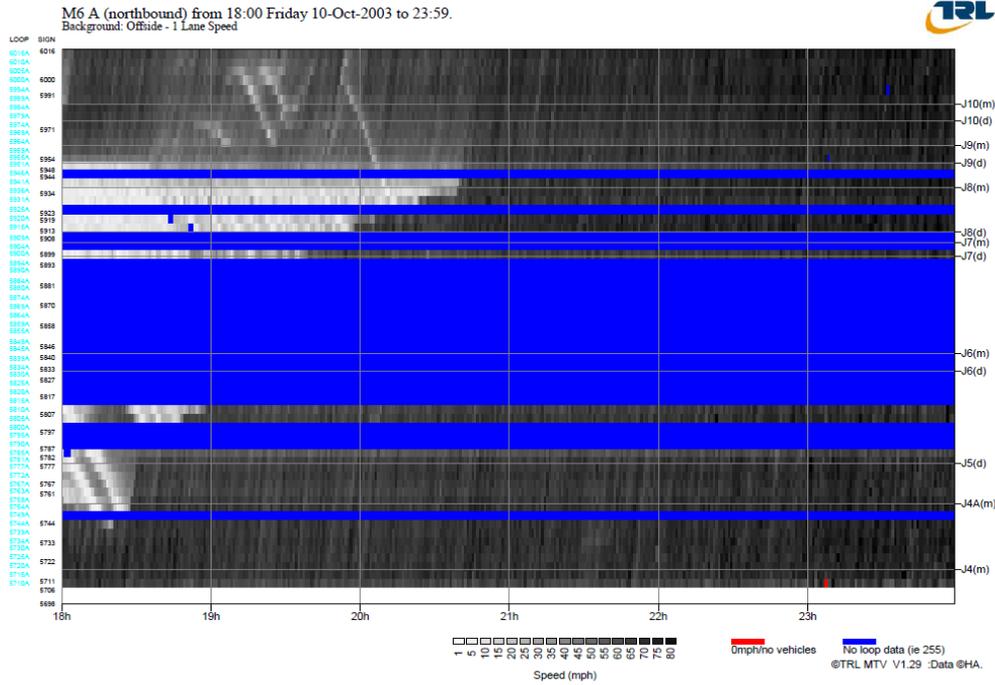Background: Offside - 1 Lane Speed

Figure 4.3 M6 Northbound MTV Plot 10/10/03 1800-2400

A short period of 'standing wave' flow breakdown occurs at the Junction 9 merge between about 0720 and 0725, but does not extend upstream. This may be caused either by a short spike in demand or some minor unreported incident.

'Standing wave' flow breakdown at the Junction 8 merge commences at around 1155. It appears that this is caused by a 'shockwave' which commences earlier at a 'seed point' just before Junction 9 diverge but the malfunctioning of loop 5946A precludes definite verification of this.

The 'standing wave' flow breakdown at the Junction 8 merge continues without any break until it clears at around 2040. The queue reaches Junction 7 diverge by 1310, but due to the unavailability of MIDAS speed data upstream of this location, it is not possible to verify how much further it extends. It may extend back as far as Junction 4A and beyond in 'shockwaves'. However, the reduced lane widths at the roadworks between Junctions 6 and 7 and the likely heavy merge at Junction 6 (A38(M)) are both likely to have contributed to this queue.

There is a 'shockwave' flow breakdown at Junction 10 diverge at around 1345 which propagates through the Junction 8 merge standing wave apparently as far back as Junction 4A (it is not possible to verify this due to the missing MIDAS sector). This may be caused by flow breakdown on the off-slip or junction roundabout caused either by an incident or queueing at the roundabout.

At around 1405 there is a 'shockwave' which either starts at the northern end of the MIDAS area or beyond. This appears to propagate back to Junction 4A. This is followed by a series of 5 'shockwave' flow breakdowns with a 'seed point' around 2 kilometres downstream of Junction 10 merge between around 1430 and 1530 with a frequency of approximately every 15 minutes. Each of these causes a short period of 'standing wave' flow breakdown at Junction 10 merge. These 'shockwaves' propagate upstream through the Junction 8 merge 'standing wave' and apparently as far upstream as Junction 4A.

The question arises as to why the 'shockwaves' are so regular. A possible explanation is that the effect of the shielding of excess demand reduces as the 'shockwave' propagates upstream and ceases to restrain flow at the Junction 10, Junction 9 and Junction 8 merges, and once it has passed through the Junction 8 'standing wave' it ceases to have any effect of restraining demand. Therefore, demand can build up to a level which causes a repeat of the 'shockwave' flow breakdown downstream of Junction 10 merge. Compared with the TRL work, the interval between 'shockwaves' downstream of M25 Junction 15 is shorter at about 10 minutes, but this is due to the Junction 15 'standing wave' being closer to the 'seed point' than is the case for the M6.

At 1620, starting with a 'shockwave' whose 'seed point' is about 1 kilometre downstream, continuous 'standing wave' flow breakdown occurs at the Junction 10 merge until 1800. After this there are no clear 'shockwaves' downstream of the merge, but flow is relatively slow. However, 'shockwaves' appear to propagate back from the Junction 10 merge 'standing wave'. It is notable that the queue upstream of the Junction 8 merge 'standing wave' becomes slower after these 'shockwaves' reach it, but the effect of the individual 'shockwaves' is mainly lost in the 'standing wave' queue.

A 'standing wave' commences at the Junction 9 diverge at around 1730, apparently started by the last 'shockwave' from the Junction 10 merge. This 'standing wave' continues intensely until around 1810, but thereafter less intensely until around 2040. The latter may just be slow moving traffic exiting the Junction 8 merge 'standing wave'. The Junction 9 diverge 'standing wave' must be caused by either congestion or an incident on the Junction 9 off slip or roundabout. There are 3 'shockwaves' from the 'seed point' 2 kilometres downstream of Junction 10 merge between 1910 and 2000, but only the last of these propagates upstream to Junction 9 diverge.

The recovery of the 'standing wave' at the Junction 10 merge at 1800 is clearly caused by the Junction 9 merge 'standing wave' shielding sufficient demand to allow recovery.

The profiles of "excess journey times" (period journey time minus the minimum) for 10/10/03 for different sections are shown in Figure 4.4.

Figure 4.4 Excess Travel Times for 10/10/2003
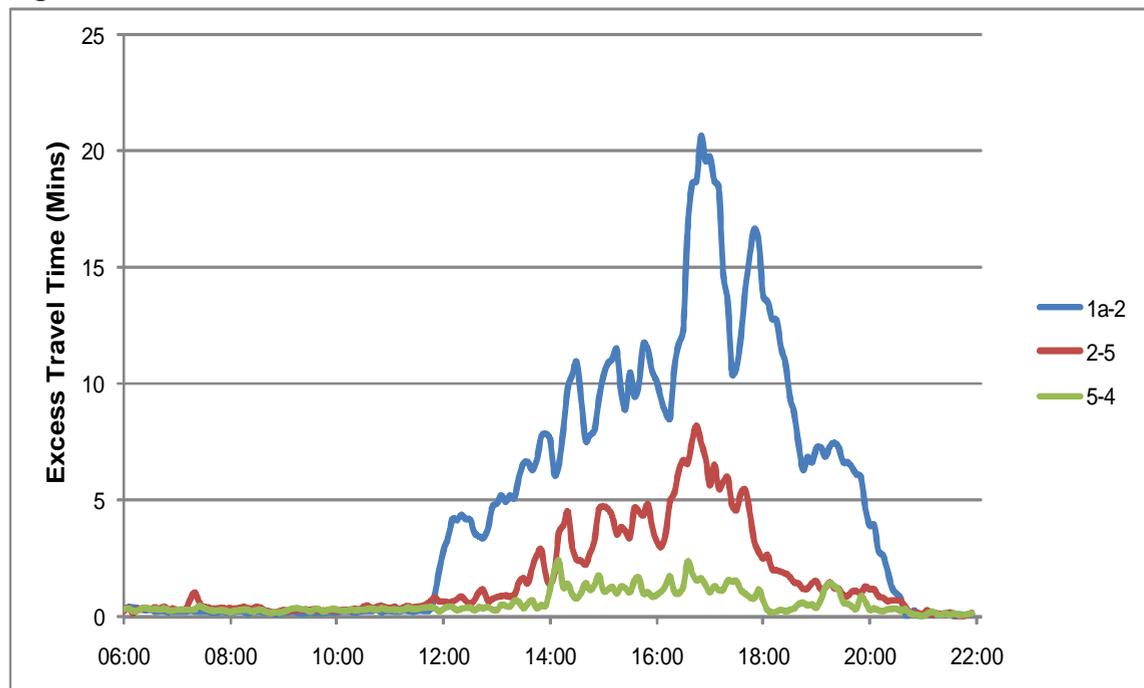
The data relates to:

- cameras 1a to 2, which covers the queue from the Junction 8 merge;
- cameras 2 to 5 which covers the queue from the Junction 10 merge back to the Junction 8 merge; and
- cameras 5 to 4 which covers the section downstream of the Junction 10 merge

This demonstrates that the 'standing waves' at the merges cause the vast majority of the delay. The delay caused by the 'shockwaves' downstream of the Junction 10 merge is small.

4.5   Flow Breakdown Types

There are two types of flow breakdown, 'shockwave' and 'standing wave'.

The 'shockwaves' occur at 'seed points' on links where capacity is effectively slightly less than upstream due to physical issues (such as gradients). The head of the 'shockwave' propagates upstream, as the moving wave shields demand and allows the recovery of the 'free flow' state downstream. 'Shockwaves' occur at regular intervals due to the shielding effect on the 'seed point' of the previous 'shockwave' being reduced as it moves upstream. Any vehicle travelling in 'shockwave' conditions will therefore only experience the flow breakdown caused by the 'shockwave' for a short period, but this may occur several times. This suggests that the delay effect of 'shockwave' flow breakdown is unlikely to be great as it does not cause queueing.

'Standing wave' flow breakdown occurs at locations where there is a bottleneck, at which demand can significantly exceed capacity. Such bottlenecks may be:

- incidents which block one or more lanes;
- roadworks which block one or more lanes;
- merges where there are high flows joining the motorway and there is no lane drop through the junction; or
- diverges where queueing on the off-slip causes tail-backs which effectively cause the loss of one or more lanes on the main line or where there is a lane drop.

For the 'standing wave' the head of the flow breakdown remains at the bottleneck point, as there is no prospect of recovery of the free flow state as long as the demand exceeds capacity. If demand significantly exceeds capacity long queues can build, which lead to long delays. In view of this the 'standing wave' is a much more important flow breakdown feature than the 'shockwave' both in terms of delay and travel time variability (TTV).

An important feature of 'standing waves' at merges is that they usually appear to be instigated when a 'shockwave' from upstream reaches the bottleneck point. This is likely to be because the 'shockwave' momentarily reduces capacity. As there is potential demand waiting on the on-slip this takes up the reduction of potential main line demand due to the shielding effect of the 'shockwave' moving upstream. Hence, free flow cannot be recovered at the bottleneck. Therefore while 'shockwaves' may not be important in themselves, they have the effect of instigating 'standing wave' flow breakdown.

The definition used for the identification of 'standing wave' flow breakdown at the M6 junctions 8, 9 and 10 merges is discussed in Section 8.2.

4.6   Implications for Modelling

For modelling 'shockwave' flow breakdown it is appropriate to use speed-density type relationships relating to link flow. Given that 'shockwaves' only have a very short term effect their effect on journey times and TTV can be measured using speed flow or speed-density relationships.

For the modelling of 'standing wave' flow breakdown (and incidents) the most appropriate approach is one based on queueing theory. However, for merges, there is the added complication that demand arises from two potential sources; the main line upstream; and the on-slip. The proportions of these demands in the queue that discharge at a point in time are dependent on their individual levels of demand and the physical layout of the merge. To understand this for the Junction 8 and 10 merges it is necessary to analyse the movements and queue lengths of both the M6 main line and the on-slips.

Given the uncertainty over timings the behaviour of individual 'shockwaves' is too complex to model. Also while they may contribute to starting 'standing wave' flow breakdown they have little effect on journey times and TTV relative to 'standing waves'. In view of this it is appropriate to predict the onset of 'standing wave' flow breakdown directly based on the demand at the bottleneck. In the case of merges this is the combination of the main line flow through the junction and the on-slip flow.

Where 'standing waves' from downstream merges extend to the head of the upstream 'standing wave' they are likely to further reduce discharge capacity at the latter. This is the case when queues from the Junction 10 merge 'standing wave' extend back to the Junction 8 merge 'standing wave'. It is necessary to model this interaction.

There is occasional 'standing wave' flow breakdown at diverges. This is probably caused by congestion or incidents on the off-slip or junction roundabout. The M6 dataset does not include any information on these issues; therefore it is not possible to model this type of flow breakdown. In view of this, for model calibration, days were avoided where there was significant 'standing wave' flow breakdown at diverges.

5 MICROSIMULATION MODEL

5.1 Previous Model

In the Arup work, as subsequently confirmed in the Stage 1 work, we had developed a S-Paramics microsimulation model which had been shown to achieve a good representation of the outturn journey time profile for a particular day. In addition, we had constructed a procedure for perturbing the input demand according to a Monte-Carlo simulation procedure (M6FRAME) and shown that this was capable of generating a convincing pattern of TTV. Although all this had been achieved by concentrating on a very small section of the overall data, it was our expectation that the main contribution to TTV was the variability of input demand.

The original Arup micro-simulation model was developed using the 2002.1 version of S-Paramics – that being the most up-to-date version available at the time. The model included hourly demand matrices for each individual movement between entries and exits for the modelled section of M6, between 06:00 and 22:00. These matrices were synthesised based on the observed entries and exits. In addition to this, the demand from each entry was fully profiled into 5-minute periods based on observed MIDAS count data. The model concentrated on the northbound carriageway only.

The matrix of movements in the S-Paramics model is shown in Figure 5.1.

Figure 5.1: Matrix of Movements within the S-Paramics Model

Destination / Origin	5 M6 J9d	6 M6 J10d	7 M54 West	8 M6 North
1 M6 J8d to J8m	🟩	🟩	🟩	🟩
2 M5 Slip	🟩	🟩	🟩	🟩
3 M6 J9m		🟩	🟩	🟩
4 M6 J10m			🟩	🟩

Four vehicle types were distinguished, as shown in Table 5.1. These are based on the length groups detected by MIDAS. Each had their own demand matrices and release profiles.

Table 5.1: Vehicle Types

Vehicle type no	MIDAS length bin definition	S-Paramics definition
1	< 5.2 m	Cars and most light goods vehicle
2	5.2 to 6.6 m	Long wheelbase light goods vehicles
3	6.6 to 11.6 m	Medium goods vehicles (OGV1)
4	> 11.6 m	Heavy goods vehicles (OGV2) and coaches

More details are provided in Working Papers 15 (Arup, February 2003) and 22 (Arup, October 2003) from the Arup study.

5.2 Model Re-calibration

The subsequent Audit carried out in Stage 1 resulted in a few minor changes and recommendations. Various modifications to the S-Paramics software have been made since the Arup work, as well as new facilities introduced. After experimenting with different versions, it was decided to use the S-Paramics 2005.1 version for the work. This gave the closest replication of the earlier results. In addition, it was advised in the audit that each scenario should be run ten times, using a different random seed for each run, with an average of these results used for calibration and implementation of the model.

In general, the audit concluded that the model was fit for purpose, but the physical network review identified that the ramp coding at J10 needed to be altered. Observations of the model in operation showed significant breakdown in the vicinity of the ramp, for which the reasons needed to be established. Additionally, a "bug" was found in the operation of the merge at Junction 10, and the required modifications led to a need for some re-calibration.

Although when using the 2005.1 version of the software (before any network updates were carried out) the total journey times from Camera 1a to Camera 4 were acceptable, it was considered necessary also to ensure that the model was showing delays in the correct locations. Hence, during the re-calibration process, each of the individual camera-to-camera sections was calibrated separately.

In the first place, it was decided to use the same day of traffic data and observed journey times as used previously – 28th January 2003. Early data analysis work had indicated this was a suitable day to use for model calibration, as no incidents had been reported.

This produced a version of the model which not only maintained the level of calibration reported previously, but in fact gave a slightly better representation of congestion, particularly in the PM Peak, in terms of the overall performance in relation to Camera 1a to Camera 4, though the actual location of delays was less well reproduced. However, an attempt to transfer the model to a different day - Tuesday, 11th February – did not produce satisfactory results: the model produced delays greatly in excess of those actually observed.

5.3   Preparation of Input Data

As noted in §3.5, the S-Paramics model development worked with 25 weekdays were identified with no incident effects and not more than 10 minutes of missing MIDAS count data. For practical purposes, this list was reduced to a limited number of days for initial calibration:

- Thursday 13th March 2003; and
- Thursday 6th November 2003.

These two Thursdays were considered to be the most typical of term-time Thursdays although both showed differing levels of delays in the critical PM Peak period.

The intention was that once the model had been calibrated against both of these independent days to a suitable level, it would then be used to run a Monday and a Friday, both of which show significantly different levels of delay to midweek days.

It should be emphasised that the task of model calibration was carried out in parallel with the data processing task, with the practical outcome that some days were chosen that were subsequently found to be inappropriate, and that unexpected problems were encountered which led to a further review of the data requirements. In spite of this, we do not believe that the conclusions would be substantially affected.

5.4   Testing

After allowing for slight model changes, assignments and calibration checks, a model referred to as the MTTV Model was produced for 13th March 2003. The output journey times were a good fit to those observed, as shown in Figure 5.2.

Figure 5.2: Comparison of Actual and Modelled Journey Times (Cameras 1a to 4) – Thursday 13th March (MTTV Model)
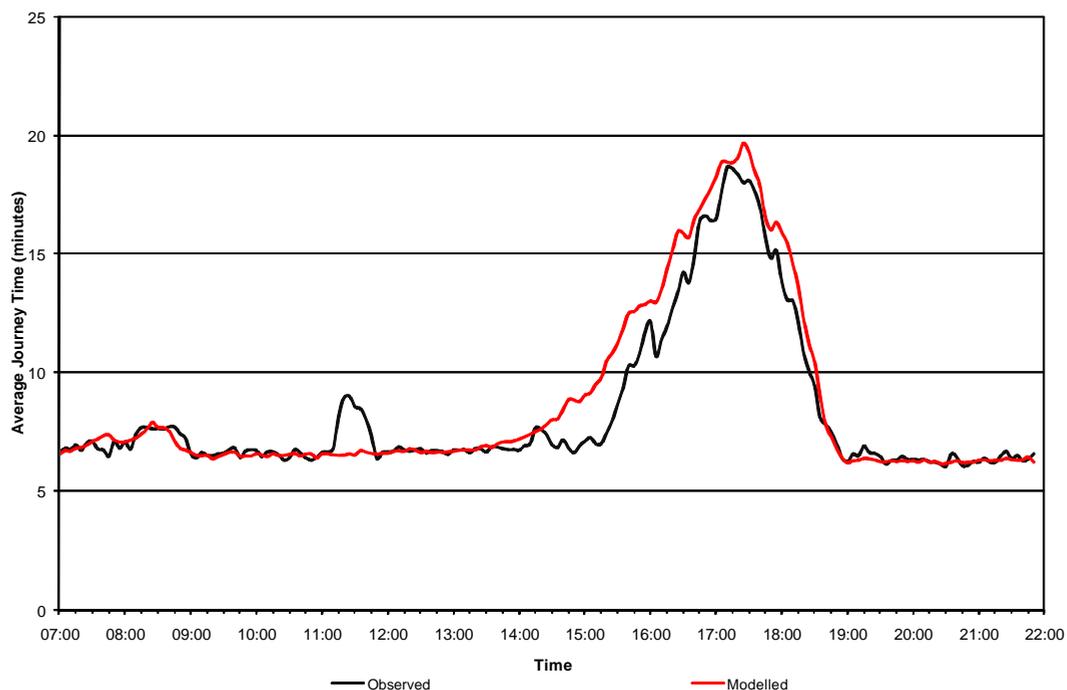
Although the model showed congestion building at an earlier time than observed, the scale of the delays was similar to that of the observations. In addition, the time and rate at which the delays dissipate was very similar to the observed data.

As with the previous calibration, care was taken to ensure that the delays were being modelled at the correct locations at the correct time. Each individual section of the study area was calibrated in its own right. It was found, however, that the correlation between modelled and observed journey times on the section relating to the M5 approach to Junction 8 (Camera 1b to 2) was relatively poor. Although the delays were of a similar scale, the time that congestion started to increase, and then in turn dissipate, was much earlier in the model than in the observed.

The MTTV model was then run using an independent data set for the alternative day of Thursday 6th November. Once more, as with the earlier example of 28th January and 11th February, the assignment of the second data set to the model was not successful. Modelled journey times were, in this case, far **lower** than those observed. For the second time, a model had been produced that closely matched travel time conditions for one particular day, but was then unable to carry this through to an independent day.

From this, and the journey time calibration along the successive sections of the M6, it became evident that a better understanding of exactly why breakdown was occurring at given locations was needed. To achieve this, MTV plots were examined for the modelled days. While both "shockwaves" and "standing waves" can be detected, it was considered more important, in line with the discussion in the previous Section, to ensure that the model operated correctly with regard to the standing waves. In any case, because the model is run ten times using random seed values, average speed outputs would tend not to show such distinct shockwaves propagating through the network, as the waves may occur at differing times in each run.

Further, to develop a better understanding of the observed and modelled patterns of congestion, a more detailed comparison of the model inputs was carried out. This was intended to clarify why certain days may experience higher levels of delays – the expectation being that these are caused by higher demands or possibly due to more subtle difference in the demand profiles in the congested PM Peak period.

The journey time comparison in Figure 5.3 shows that up until approximately 17:15 the two days (13th March and 6th November) behaved very similarly. But from this point one day suffered a further growth in congestion whereas the other experienced a reduction.

Figure 5.3: Comparison of Actual Journey Times (Cameras 1a to 4) – Thursday 13th March and Thursday 6th November
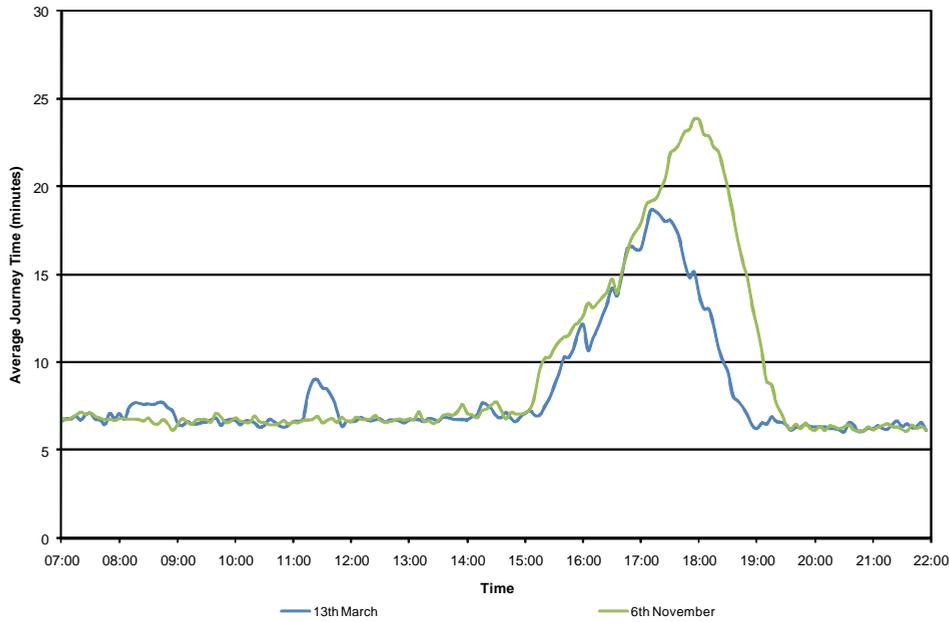
On deeper investigation, it transpired that the key difference between the observed journey times for the two days was the congestion within the Camera 3 to Camera 4 section in the vicinity of Junction 10. Figure 5.4 illustrates this difference in journey times.

Figure 5.4: Comparison of Actual Journey Times (Cameras 3 to 4) – Thursday 13th March and Thursday 6th November
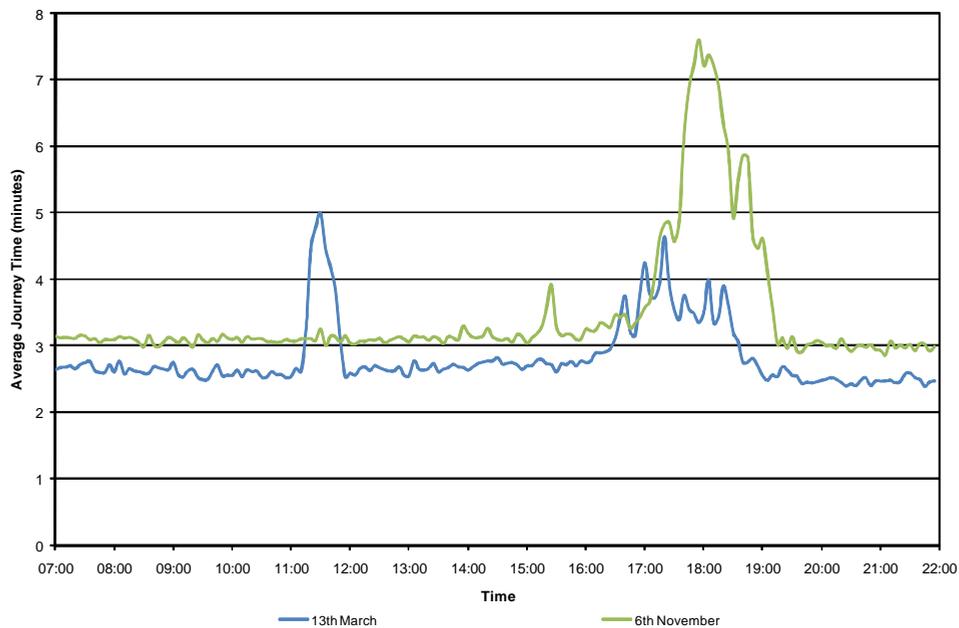

On 13th March, shortly after 17:15, the delays began to diminish giving a gradual return to free-flow conditions, albeit with the occasional spike due to the shockwaves developing downstream of Junction 10. But on 6th November journey times continued to rise consistently until approximately 18:00, peaking at roughly 7.5 minutes – 3 minutes more than during the March day peak.

To try and explain these very different patterns of delay, a comparison of the assigned demand at Junction 10 was made. It would be expected that at some time after 17:15, the demand on 6th November would be noticeably higher, causing the continuing development of congestion.

Now of course while we have the in- and outflows at the various entries and exits to the M6 section, we do not have the demand at any particular point. As noted later in Section 7, the inflows themselves are not true "demand", since they may be constrained by upstream queueing. For the purpose of developing an appropriate comparison, we have calculated the "demand" on both the mainline and the on-slip at J10 throughout the PM Peak period assuming free-flow conditions from the point of entry. This allows for the time taken to reach the required point on the network.

On this basis, Figure 5.5 shows a comparison of the demands (termed "Assigned Demand Flows") through J10 for the two days of interest. The solid lines relate to the mainline flows, and the dashed line to the on-slip. The flows are measured as vehicles per 5-minute period across all lanes: the mainline section has 3 lanes.

Figure 5.5:   Comparison of Assigned Demand Flows through Junction 10 – Thursday 13th March and Thursday 6th November: pm peak
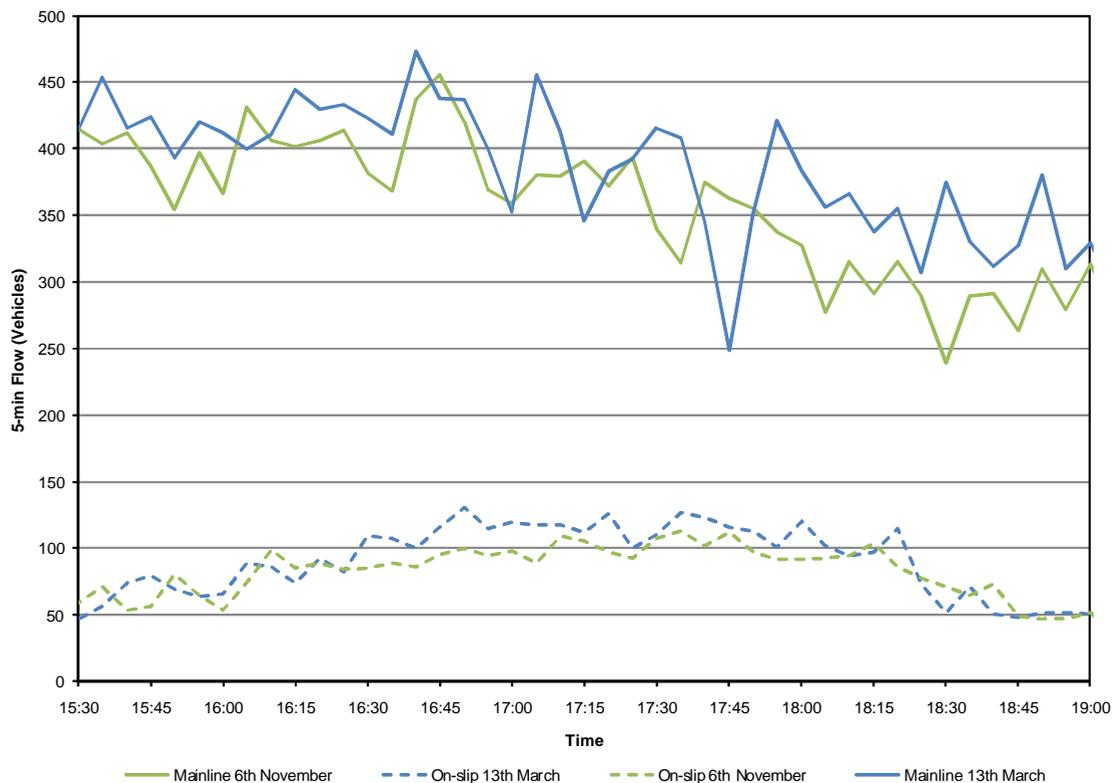
Although, as noted, the flows plotted here are the observed flows, and will not represent true demand when congestion upstream is present, the levels of delay experienced throughout the network are similar on both days up until 17:15. Hence, there is no reason to think that the relation between demand and flow was dissimilar on the two days. In fact, the on-slip demand has no intermediate junctions to traverse, so in that case demand **will** be equal to flow.

Due to these flows being based on 5-minute periods, the values can fluctuate greatly, especially on the mainline where consecutive periods can have very different demands. It was apparent, however, that for the majority of the PM Peak the demands on the mainline were higher on 13th March, apart from an individual trough at about 17:45. It can be seen that the demand on the on-slip on 13th March appears consistently **higher** throughout the PM Peak.

It can also be seen that the demand on 6th November, at the time when congestion grew rapidly at about 17:30, was in fact nowhere near as high as earlier in the PM Peak. The demand at 17:45 on 6th November was approximately 470 vehicles per 5-minute period (equivalent to 1880 vehicles per lane per hour). At 16:45, however, the demand was roughly 530 per 5-minute period (or 2120 vehicles per lane per hour). At this time on 13th March the demand peaked at 570 per 5-minute period (or 2280 per lane per hour).

It would seem that for our model to reproduce journey times for the two independent days, it would have to produce much higher levels of delay when the demands that were being input were in fact lower. Clearly, this presents a problem. We therefore made further effort to try to rationalise the extra delays.

The MTV plots for 13th March show that the delays through Junction 10 were triggered by shockwaves which were initiated downstream towards Junction 10a, heading back upstream through Junctions 10 and 9 before reaching the standing waves at Junction 8. The speed outputs from the model, however, showed that the delay at Junction 10 was being caused at the merge itself (suggesting an under-estimation of capacity at the merge). The model output shows a standing wave at the Junction 10 merge; the delays associated with this have no interaction with any downstream congestion. When the total demand reached an equivalent of approximately 2000 vehicles per lane per hour (500 vehicles per 5-minute period), breakdown was apparent through the merge at Junction 10 in the model – causing a standing wave.

By contrast, the demand on 6th November very rarely reached this level of 500 vehicles per 5-minute period. Hence the delays generated in the model were minimal through Junction 10, with journey times only deviating from free-flowing conditions for a short period in the PM Peak. This suggests that factors other than simply the volume of traffic were responsible for the breakdown through Junction 10.

Analysis of HGV proportions showed no significant difference between the two days. Another possibility is that a subtle difference in the level of weaving for vehicles approaching Junction 10a was responsible. This could be caused by a larger proportion of those vehicles exiting the M6 at Junction 10a having originated from further upstream on the M6, rather than Junction 10, thus resulting in a higher need for lane changing downstream of Junction 10. Unfortunately, detailed origin-destination data is not available: the input demand matrices have to be synthesised from counts at each junction.

It had been expected that the key element of the variation in observed journey times was due to fluctuations in demand. However, from the analysis carried out, it would seem this was not the case. This puts the emphasis more on the supply variability.

5.5 Random Effects Modelled in S-Paramics

According to documentation provided by SIAS (SIAS, 2010), the random effects modelled in S-Paramics are as follows.

5.5.1 Release Distribution

The two key inputs for traffic demands in S-Paramics are the demand matrices themselves and also the demand profiles. In the case of the MTTV model, the demands were determined by hourly matrices, which then had associated demand profiles to determine the demands in intervals of five minutes.

However, within these five-minute periods, the exact release time of each vehicle is random and is determined by one of two algorithms; 'Precise' or 'Stochastic'.

The Precise release will generate within plus or minus one vehicle of the demand matrix for each origin-destination movement. The exact times of release for each vehicle within the five-minute period are random, and will vary from run to run, dependent on the random seed used.

The Stochastic release gives some statistical variation in the number of vehicles released in each five-minute interval. The probability of a release each second is calculated and a linearly random number generated every second during the five-minute interval to determine whether a vehicles is released that second or not.

The average release for each five-minute period will tend towards the inputted value, but in any given assignment the actual release from Zone i to Zone j will be distributed around the input value.

This Section summarises Appendix B (S-Paramics Release Distribution) of (SIAS, 2010). In relation to these options, the 'Precise' release method was used in our modelling. This is the default setting, and, on the basis of limited testing, it was judged that the alternative 'Stochastic' release would have no significant influence on the model's outputs.

5.5.2 Behavioural Variability

Each vehicle in a S-Paramics simulation carries two driver attributes – 'awareness' and 'aggression' – where the values of these attributes control the reaction of the driver to road conditions and the presence of other vehicles.

Each vehicle is assigned a value for each of these at random when released (from a pre-defined distribution), with these values being maintained throughout the entire trip of the vehicle.

By default, the awareness and aggression values are determined by a normal distribution about a mid point, of which SIAS has empirical evidence that this provides good results.

Each of the awareness and aggression attributes have 8 possible values that can be assigned to a vehicle. Therefore, the combination of the two gives rise to 64 distinct possible vehicle behaviours.

The aggression and awareness of any given vehicle influences its behaviour in a number of situations throughout its trip. These are as follows:

- Speed – determined primarily by aggression;
- Headway (including when approaching a ramp) – determined primarily by aggression;
- Reaction times – determined primarily by awareness;
- Overtaking on an opposing lane – determined primarily by aggression;
- Overtaking (including moving back to the nearside lane) – determined primarily by awareness;
- Lane changing – determined by both awareness (in perceiving a gap) and aggression (when accepting the gap);
- Courtesy slowing (to allow another vehicle to make an urgent lane change – determined primarily by awareness; and
- Lane changing (in advance of a downstream hazard) – determined by both awareness and aggression.

Aggression values are also used to determine a vehicle's lane choice on a carriageway, especially on highway links, when approaching a diverge and also when clear of any junction; with more aggressive vehicles being more likely to utilise the outside lanes in their possible lane range.

The gap acceptance of vehicles on the minor arms of junction is indirectly controlled by aggression and awareness as a result of the speed and headway of the vehicles as mentioned above.

This Section summarises Appendix C (Behavioural Variability) of (SIAS, 2010). The default values have been used in our modelling.

5.5.3 Route Choice

A further cause of variability in a model is due to route choice perturbation. This primarily applies to a random selection of the link a vehicle chooses at a junction based on its perception of the relative route cost of each choice.

Links may also be coded as 'major' or 'minor', and vehicles determined as 'familiar' or 'unfamiliar' – both these values inputs being determined by the modeller. The perceived costs of each type of link differ for each vehicle class.

However, in a highway corridor model such as the MTTV Model, this is not applicable and has no influence on the behaviour resulting in flow breakdown.

5.6 Conclusions for Demand-Based TTV

It had been hoped that a single micro-simulation model could be developed that would reproduce the delays evident on any given day. However, with only the random effects discussed in Section 5.5 being modelled, it was not possible to produce either (a) very different modelled journey times with similar input flows, or (b) very similar modelled journey times but with significantly varying input flows. It therefore became clear that any model calibrated against journey times for a given day would have difficulties in giving a good representation of the journey times on any other independent day.

In the cases that were analysed in detail, it would seem that the delay observed did not correlate with the level of flow. In modelling terms this made the task impossible. A network model could not be expected to output higher delays when the key demand inputs were much lower. This suggests that the role of demand variation in outturn JTV may be less than previously expected, and that more attention is required to the contribution of "supply effects", in particular the occurrence of flow breakdown. While S-Paramics is capable of reproducing flow breakdown, it does this on a largely deterministic basis, when a critical flow is reached. As we shall see in Section 8, the evidence is that while the **probability** of breakdown increases with the level of demand, breakdown may occur over a wide range of demand, and it is this stochastic variation which has a major impact on TTV.

In the light of these conclusions, it was decided not to proceed further with the development of a microsimulation model based on S-Paramics for the purposes of the study. We made a detailed report of our findings (Hyder, May 2010), which we provided to SIAS, the owners of the S-Paramics software. They delivered a helpful response and we have reproduced in Section 5.5 what we believe to be the key information from their response. We are not aware, however, of any appropriate mechanism whereby the stochastic nature of flow breakdown can be satisfactorily handled within the current S-Paramics software.

6 OUTLINE OF AGGREGATE MODEL

6.1 The Rationale for an Aggregate Approach

In the earlier Arup work, the appropriate modelling approach for predicting TTV was discussed at some length (§8.3). It is worth citing the relevant text:

8.3.3 *The challenge is to construct a model which is sufficient to deliver an estimate of journey times under different profiles of demand, both for the main demand and the merging demand. A key question was whether something satisfactory could be achieved at a relatively "aggregate" level, or whether we would need to move to a microsimulation approach, allowing explicitly for variation in "gap acceptance".*

8.3.4 *There was an understandable concern that, despite the advantages in terms of reduced complexity for using an aggregate analytical approach, gradual increases in complexity might mean that we would reach a stage where no further progress could be made. On the other hand, we were not aware of any conclusive evidence that a microsimulation approach would be able to reproduce the key feature of breakdown as enumerated above.*

8.3.5 *[....] some initial tests with a simple spreadsheet model, as an extension of an earlier bottleneck model, showed that it was possible to simulate two key aspects of breakdown (capacity and speed reduction) in line with some of the empirical evidence. Because of its close relationship to the simple bottleneck, this will also generate the "loop" relationship between the mean journey time and the standard deviation of journey time or the coefficient of variation of journey time. Further modifications can be made to deal explicitly with the change in demand associated with the merge. It is relatively straightforward to combine this with an allowance for stochastic variation in demand and "capacity", along the lines discussed earlier. We therefore took the view that this was a fruitful approach.*

8.3.6 *Nonetheless, we considered it prudent to adopt a two-pronged strategy, involving parallel developments of an aggregate analytical approach and a microsimulation approach, while maintaining consistency between the two as far as possible.*

Although we commenced the current project with the view that the microsimulation work appeared more promising, the conclusions of the previous section, and the subsequent abandonment of the S-Paramics model, meant that the Aggregate Model needed to be fully developed for the purposes of the study.

In this section, we describe the concept of the model, and then, in the following sections, its detailed development.

6.2 Description of the Aggregate Model

The aim of the model is to serve as a tool for the prediction of travel time variability (TTV) on the northbound section of the M6 between Junctions 8 and 10A. Two key components are considered necessary for this: a stochastic variation in demand at different times of day, and a stochastic prediction of flow breakdown and its consequences, or, under non-breakdown conditions, a stochastic speed-flow relationship. The combination of these stochastic components should be sufficient to provide a good prediction of the variation in travel times at different times of day.

Both components have been based on analysis of the data collected for the motorway section in 2003. The model is/should also be able to be/ validated, both in terms of mean travel times and the variation in mean times, against the 2003 data.

The resulting "aggregate" modelling of motorway flow breakdown simulates traffic flow breakdown and estimates vehicle travel time at a macroscopic level, instead of detailed vehicle movement modelling. This approach identifies the key quantities that are of interest, such as average journey time and journey time variability whilst ignoring other factors, such as the OD matrix, lane changes and vehicle operating characteristics which are important in the micro-simulation modelling. The Aggregate Model provides a simple method with very low computational burden that is able to capture flow breakdown (associated with "standing waves") and its impact on travel time and travel time variability.

The development of the stochastic demand component is described in Section 7. Briefly, based on statistical analysis, a number of distinct "day groups" have been defined, and for each of these, the mean profile over the day of demand entries and exits has been estimated, together with the observed variation around these profiles. This constitutes the demand input to the Aggregate Model.

With these inputs, the model then has to account for the progress of the traffic demand through the limited stretch of motorway, including the possibility of flow breakdown, and to estimate the travel time between different points. By repeated sampling from the demand distribution, together with the stochastic supply effects, the TTV can be estimated.

We begin by outlining the Aggregate Model for a single link. It can be shown that the structure, while an approximation, is compatible with the traffic flow theory for the propagation of traffic along a single link, in continuous time and space. Following that, the proposed treatment of breakdown is introduced for a single merge.

Having set out the basic properties of the system, we discuss the appropriate depiction of the links for the entire section.

6.2.1 Traffic propagation under non-breakdown conditions

The model operates in successive intervals of duration I, and it has been decided to set I to 5 minutes. In addition to this temporal aggregation, we also assume spatial aggregation, in that we imply (under normal circumstances) some homogeneity in the flow along the link. This allows us to relate the prevailing speed to the total traffic on the link. Clearly, this introduces some constraints on application, related to the length of the link: we would not wish to apply the formula to a long link with widely different flows between the first and second halves. Essentially we have relatively short links in mind, and these also need to be compatible with the temporal aggregate units assumed.

The model works in terms of vehicles, and no allowance is made for variations in vehicle composition.

In any one period r, we define a constant entry rate d_r . We write Q_r for the cumulative entries up to and including period r. Hence, we can write ΔQ_r, the change in cumulative entries during period r, as:

$$\Delta Q_r = d_r . I \tag{1}$$

Similarly, we define $\Delta \Omega_r$ as the change in cumulative exits during period r. This gives:

$$\Delta S_r = S_r - S_{r-1} = \Delta Q_r - \Delta \Omega_r \tag{2}$$

where S_r represents the vehicles on the link at the end of period r.

Under normal operating circumstances, we assume that a uniform speed v_r applies throughout the link during period r. This allows us to set out the equation for W_r, the time to traverse the link in period r:

$$W_r = l/v_r. \tag{3A}$$

where l is the link length

We now use this to calculate $\Delta \Omega_r$, the number of vehicles leaving the link in period r. The number of vehicles already on the link at the start of period r is S_{r-1}. If $W_r <$ I, where I is the interval length (5 minutes), then all these vehicles will leave the link in period r. In addition, a further d_r.I enter the link during period r, i.e. during the time interval [(r–1).I, r.I]. Since these vehicles require W_r time to clear the link, those entering after (r.I – W_r) will still be on the link at the end of period r. Assuming uniform spacing, this means that of the "new" demand, only a proportion will leave. Hence, as long $W_r <$ I, the output in period r is given as

$$S_{r-1} + d_r . (I - W_r).$$

The calculation becomes more complicated when $W_r >$ I. In this case, clearly none of the new demand will clear the link. As for those already on the link, we have to calculate how many of them will clear it, and this depends on the distance. The maximum distance that can be traversed in the interval I is I.v_r. Hence only the fraction of S_{r-1} that are less than or equal to this at the start of the period will leave. Again assuming equal spacing, this proportion can be estimated as

$$I.v_r/l = I/W_r.$$

Hence, we can summarise this as follows:

If $W_r = l/v_r <$ I then $\Delta \Omega_r = S_{r-1} + d_r . (I - W_r)$, else $\Delta \Omega_r = S_{r-1}. I/W_r$ (4A)

To implement this system, we require a specification for v_r. One approach would be to assume that in period r, the speed v_r depends on the density at the end of the previous period, ie $v_r = f[S_{r-1}/(l.n)]$ where n is the number of lanes. In the event, after some consideration of the available data, a more standard speed-flow relationship was implemented.

To allow for supply variability under non-breakdown conditions, an appropriate stochastic element is added, derived from an analysis of the M6 data, so that:

$$v_r = f(S_{r-1}/(l.n)) + \varepsilon^v_r \tag{5A}$$

For the starting conditions, we assume that in the initial period r = 1, $S_0 = 0$, so that the link begins in an empty condition (implying free-flow speed).

These five equations (1, 2, 3A, 4A and 5A) define the dynamics of the single link system, under non-breakdown conditions

Motorway Travel Time Variability—Final Report
Hyder Consulting (UK) Limited-2212959   **Page 46**
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6.2.2　Dealing with Flow Breakdown Positions

We now go on to discuss the possibility of flow breakdown. Based on the M6 evidence, it will be assumed that this is (primarily) associated with merging. In other words, we move on from the single link conditions to discuss – in the first place – the merging of two links, as illustrated in Figure 6.1.

Figure 6.1 Merging of two links
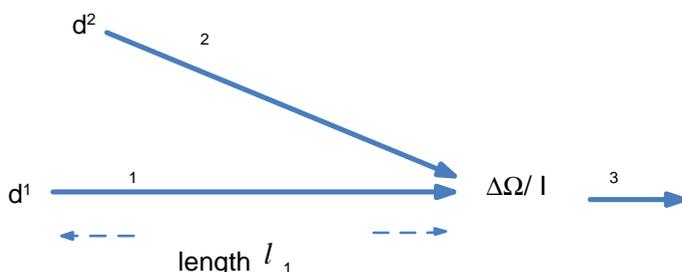
The   diagram shows two links j (1 and 2) each with input demand d^j and length l_j converging on a third link (3). The combined output from links 1 and 2 ($\Delta\Omega = \Delta\Omega^1 + \Delta\Omega^2$) becomes the input to link 3 (the division by I is to bring the quantity back to units of veh/h).

Hence, the linking equation to transfer the demand from the upstream link becomes:

$$d_r^{\,3} = \left(\Delta\Omega_r^{\,1} + \Delta\Omega_r^{\,2}\right)/I \tag{6}$$

The occurrence of breakdown is assumed to be a stochastic variable based on this demand, and we write:

If $d_r^{\,3}$ > C$_3$ + $\varepsilon^{B3}$$_r$, then breakdown $\tag{7}$

The values of C and the appropriate distribution for ε^B have been derived from analysis of the M6 data: they are specific to the particular merge link, and are discussed in Section 8.

As long as breakdown does not occur, the merge link (3) continues to follow the basic dynamic system given in the previous section. However, once breakdown occurs, the output flow $\Delta\Omega^3_r$ from the merge link is set to I.(g$_3$ + ε^{Q3}_r), where g is the average "queue discharge flow" (QDF) in veh/h, and ε^Q is a random variable. Once again, the values of g and the appropriate distribution for ε^Q have been derived from analysis of the M6 data: they are specific to the particular merge link. Again, this is discussed in Section 8. Thus, under breakdown, equation (4A) for $\Delta\Omega_r$ is replaced by equation (4B):

$$\Delta\Omega_r = I.(g + \varepsilon^Q_r) \tag{4B}$$

With this substitution, equation (2) for S$_r$ can continue to be used.

In practice, of course, traffic may block back into the contributing links 1 and 2. From the point of view of the delay calculation, we can probably ignore this as long as we correctly calculate the time W to clear the link. However, we also need to consider the implications for flow breakdown, and these are discussed later.

At the start of period r, the traffic on the link will be S$_{r-1}$, and under breakdown conditions the "service" time for each vehicle will be I/$\Delta\Omega_r$ hours. Hence, for the first vehicle entering the merge

link, the time to clear is $S_{r-1}.l/\Delta\Omega_r$ hours. Since the distance is l, the implied speed is $l.\Delta\Omega_r/(S_{r-1}.l)$.

Hence, under breakdown conditions, we replace equation (3A) by equation (3B):

$$W_r = S_{r-1}.l/\Delta\Omega_r \qquad\qquad (3B)$$

and equation (5A) by equation (5B):

$$v_r = l.\Delta\Omega_r/(S_{r-1}.l). \qquad\qquad (5B)$$

Breakdown is assumed to end when the speed given by equation (5B) ceases to be less than that implied by equation (5A).

6.2.3 General summary of model

Overall, the system can be summarised according to Table 6.1, relating to a single period (the arrows indicate the different direction of causation among the three equations for v, W and $\Delta\Omega$ under breakdown and non-breakdown conditions).
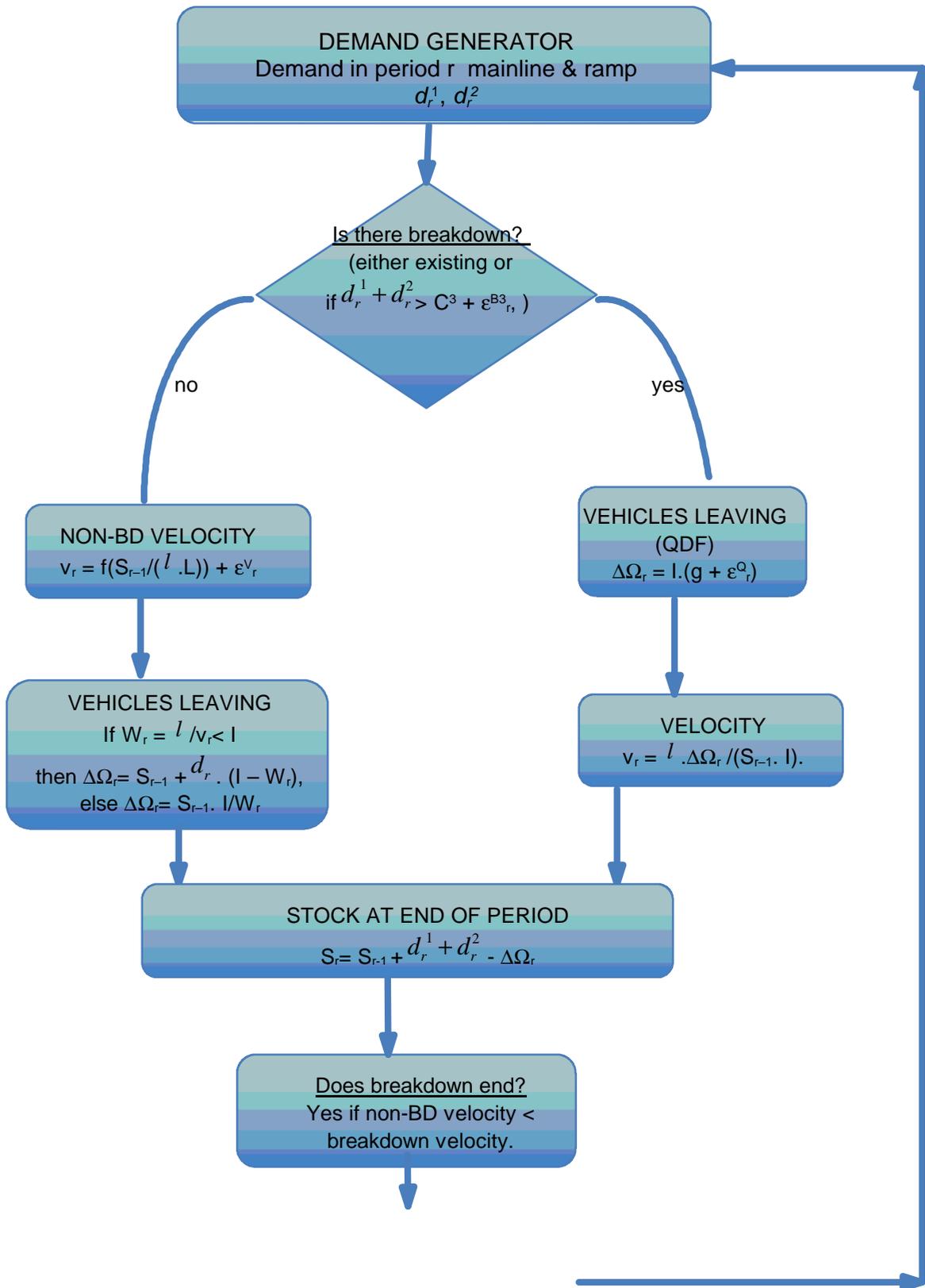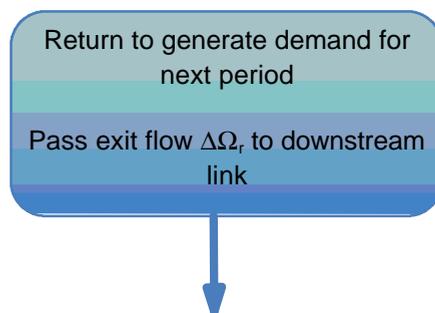
Table 6.1 General Summary of Model

new demand: $\Delta\Omega$	Eq 1	
test for breakdown	Eq 7	
	No Breakdown	*Breakdown*
link speed: v	Eq 5A	Eq 5B
time to clear link: W	Eq 3A	Eq 3B
outflow from link: $\Delta\Omega$	Eq 4A	Eq 4B
test for end of breakdown	$v_B > v_{NB}$	
traffic on link: S	Eq 2	

While the breakdown possibility **could** be applied to all links, we assume for practical reasons that in the M6 context it can generally be confined to the merge links. Of course, breakdown can also be caused by congestion on an off-slip, resulting in the left-most lane on the main carriageway being blocked.

The system outlined in Table 6.1 can also be illustrated by a general flow diagram, as shown in Figure 6.2.

Figure 6.2 General Flow Diagram Illustrating the Model Logic

DEMAND GENERATOR
Demand in period r mainline & ramp
d_r^1, d_r^2

Is there breakdown?
(either existing or
if $d_r^1 + d_r^2 > C^3 + \varepsilon^{B3}_r$,)

no

yes

NON-BD VELOCITY
$v_r = f(S_{r-1}/(l .L)) + \varepsilon^V_r$

VEHICLES LEAVING
(QDF)
$\Delta\Omega_r = I.(g + \varepsilon^Q_r)$

VEHICLES LEAVING
If $W_r = l /v_r < I$
then $\Delta\Omega_r = S_{r-1} + d_r . (I - W_r)$,
else $\Delta\Omega_r = S_{r-1}. I/W_r$

VELOCITY
$v_r = l .\Delta\Omega_r /(S_{r-1}. I)$.

STOCK AT END OF PERIOD
$S_r = S_{r-1} + d_r^1 + d_r^2 - \Delta\Omega_r$

Does breakdown end?
Yes if non-BD velocity <
breakdown velocity.

Return to generate demand for next period

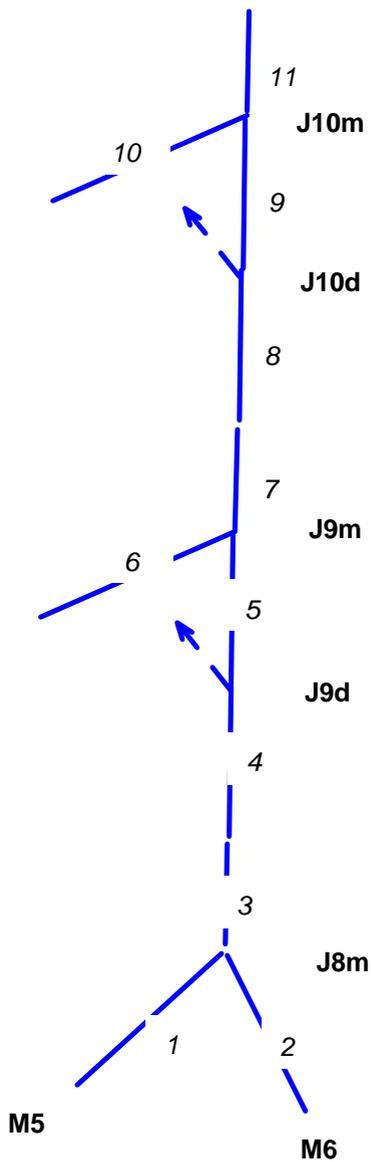Pass exit flow $\Delta\Omega_r$ to downstream link

6.2.4 Extension to cover the whole motorway section

Based on the foregoing, and with reference to Figure 2.1, the link representation for the entire section is illustrated in Figure 6.3.

Figure 6.3 Link Representation for the whole motorway section

The rationale behind the "merging links" 3, 7, 11 is that they represent the locations of flow breakdown. Each of these merging links has two "feeding links" (1+2, 5+6, 9+10). The remaining links (4 and 8) are included to denote the change in volume as traffic leaves at J9 and J10. On grounds of practical implementation, some further simplifications have been made: the merging links are treated as notional, with zero length, so that the breakdown is assumed to occur at the point of merging. In practice, therefore, 3 and 4 are treated as a single link, as are 7 and 8.

For validation of the system in terms of travel times, comparison needs to be made with the ANPR data, based on the camera locations. In relation to the figure, cameras 1a and 1b are both somewhat south of the entry points for links 1 and 2: camera 2 is about 300 metres north of J8m, camera 3 is midway along link 5, while camera 5 is about 300 metres north of J9m, and camera 4 is about 3.8 km north of J10m. On grounds of simplicity, it has been decided to concentrate on the key 1a to 4 movement, and hence link 2 is assumed to start at Camera 1a, and link 11 ends at Camera 4. This avoids the need for interpolation/extrapolation of the predicted link times.

The system is connected by means of the appropriate "linking" equations having the general form of equation (6). In general the modelling proceeds in the direction of flow, so that the links can be dealt with in the numerical order shown in the Figure above. However, an exception must be made for when flow breakdown occurs, in case the breakdown condition needs to be transferred to an **upstream** link. We now go on to discuss this possibility.

6.2.5 Backwards propagation of breakdown

Breakdown will initially be detected on a merge link on the basis of the overall demand from the two feeder links. When breakdown occurs, the feeder links will initially remain in non-breakdown state, but a further test is carried out to determine the extent of blocking back.

Two possibilities have been put forward for this test. One is essentially based on a critical density for the merge link, which can be translated into a test for the number of vehicles on the link, S_r. The alternative method is based on calibrated speed-density relationships using M6 data, separately for the J8 merge and the J10 merge. Given the prevailing speed v_r, this is input to the appropriate speed-density relationship to give a prediction of density VLK_r (veh/lane-km), and the extent of blocking back is then calculated as $S_r /(n * VLK_r)$ where n is the number of lanes, and is tested against the link length l.. Note that, in either case, this allows for the possibility of breakdown being "exported" progressively to upstream links.

The test should be carried out for the conditions at the end of the period r. Once the test is met, then both feeder links are also set into the breakdown state for the following period (r+1), and the **current** overall QDF for the merge link is allocated in a pre-defined ratio λ between feeder links for the next period. Thus, reverting to the original merge example (links 1, 2 and 3), we have:

$$\Delta\Omega_{r+1}{}^1 = \lambda^1.\Delta\Omega_r{}^3; \quad \Delta\Omega_{r+1}{}^2 = \lambda^2.\Delta\Omega_r{}^3; \quad \lambda^1 + \lambda^2 = 1 \tag{8}$$

Note that it is proposed that breakdown on a feeder link only terminates when the conditions on the **merge** link are such that breakdown terminates (ie when the speed given by equation (6B) ceases to be less than that implied by eq (6)). Of course, it is possible that this condition on the speeds may be met earlier for one of the feeder links. In this case, however, the outcome is a reallocation of the λ factors to reduce the proportion of allocated QDF on that feeder link so that it is equal to the demand. However, an alternative approach would be to allow upstream links to be progressively removed from the breakdown state. Again, these variants will need to be tested.

Overall, in modelling a set of links there are two ways in which one link affects an adjacent link.

The entry demand in link j in period r is generated by the exit flow at link(s) j-1 in the period r (equation 7).

If there is a breakdown in flow at the merge of link j then the delayed vehicles may start to affect the exit flow of link(s) j-1. If the vehicles in link j exceed some (empirically determined) density then the exit flow on link(s) j-1 is restricted to that occurring at the exit of link j. For reasons of temporal sequencing, this can only be achieved in the subsequent period (r+1).

6.3   Inputs for the Aggregate Model

The model has been implemented in the MATLAB software. Apart from the code which connects up the various equations and links, the model requires the following inputs:

- mean profile of daily demand (by day of week, by entry/exit);

- the variation in this profile (by day of week, by entry/exit);

- the parameters of a breakdown probability function (BDF) by merge;

- the parameters of a stochastic queue discharge function under breakdown conditions (QDF) by merge; and

- the parameters (mean and variance) for a speed-flow relationship under non breakdown conditions).

The derivation of demand profiles (including their variation) is the subject of Section 7. The recommended parameters for BDF, QDF and non-breakdown speed-flow are shown in Section 8. This is followed by Section 9 which validates the model against observed values of travel time and its variability.

7   DEMAND ANALYSIS

7.1   Requirements

The Aggregate Model requires inputs of travel demand (all vehicles) for each 5 minute interval over the period 0600-2200, and for the 4 entry points (M5, M6, J9 and J10). In addition, it requires the demand leaving at J9 and J10, at the same level of detail. The demand left on the M6 is implicitly destined for J10-10a: while MIDAS data is available for this, in practice it can be used for validation, and has not been further analysed.

The demand input to the Aggregate Model is stochastic, and it is therefore necessary to provide (in principle) a distribution for each item. In practice, this is interpreted as the mean and the standard deviation, plus, where relevant, some indication of the appropriate form of the distribution.

Very little previous work exists on the variability to be found in mean demand. What evidence does exist suggests that the pattern of variability is likely to be complex. The coefficient of variation (which is typically used as a metric) can vary by day of week, hour of day and other factors. In the context of a simulation of a section of motorway the data of critical importance is the demand and its variability at the entrances to the system of interest. The variability in demand at the exits is also of interest.

The required input data has been derived by a careful analysis of the available M6 MIDAS data. The data processing phase of the project has provided flow data on the 4 northbound entries and 4 exits which comprise (a) the slip roads at M6 junctions 8 to 10A and (b) the M6 at each end of the section for which data was collected. A key principle for the investigation of travel time variability (TTV) is that it should relate to unpredictable variation. Hence the data has been analysed with the intention of isolating identifiable patterns – in particular between days of the week. Generally, distinctions should only be made if they can be statistically justified.

While data from the whole period has been assembled, it has been considered necessary to exclude periods during which incidents were identified which affected the journey times on M6 Junction 8 to 10A as these would affect the demand profile by time period. In addition, the data has needed to be scanned to ensure that the flow figures do represent demand (i.e. queueing is absent), and for other anomalies.

The analysis of the assembled data has addressed the following issues:

- For mean demand profile the appropriate day groups to adopt (e.g. weekends, Mondays to Thursdays, Fridays, school holidays) and whether a distinction is needed between entries (and exits).

- For variability in demand (defined initially by the coefficient of variation) the factors that influence its level (e.g. time of day, hour of day, entry, exit)

- Whether there is correlation between time periods (e.g. higher than usual demand is followed by lower than usual demand).

- Whether there is correlation between entrances (and exits).

- The shape of the variability function

Classical statistical inference techniques using t and F-tests (based on Analysis of Variance [ANOVA]) can be used to determine significant differences between factors and whether correlation is significant.

7.2　Demand Model

The demand model is the product of the mean total travel for the whole day (0600-2200) and a mean demand profile representing the proportion of the total occurring in each given time interval t (e.g. 5 minutes). In practice, both elements (the mean daily demand and the mean profile) are subject to variability. The Aggregate Model will sample from the information provided by the demand model.

Thus we can write:

$$d_t^{if} = D_*^{if}\, p_t^{if}$$

where

d_t^{if}　is the sampled demand for a particular day for entry i, day group f and time interval t

D_*^{if}　is the sampled total daily demand for entry i, day group f,

p_t^{if}　is the sampled proportion of total demand for a particular day for entry i, day group f which occurs in time interval t

Because of the temporal dimension for p_t^{if}, we can also express it as a vector $\mathbf{p}^{if}$.

To denote the stochastic nature of both D_*^{if} and $\mathbf{p}^{if}$, we can write:

$$D_*^{if} = g(\overline{D}_*^{if}, \varepsilon^{if})$$

where

$\overline{D}_*^{if}$　is the observed mean total daily demand for entry i, day group f,

ε^{if}　refers to a stochastic variable

and

$$\mathbf{p}^{if} = h(\overline{\mathbf{p}}^{if}, \eta^{if})$$

where

$\overline{\mathbf{p}}^{if}$　is the observed mean profile vector for entry i, day group f,

η^{if}　refers to a stochastic vector over the time periods t within the day

How these two stochastic functions combine (additive, multiplicative) is to be determined. Also to be determined is the level of detail required both for distinct entries (i) and day groups (f). As the periods of daytime and flow breakdown are our interests (also the requests inputs from the Aggregate Model), all the data manipulation and analyses are based between periods 0600 and

2200 only. It was also agreed to ignore weekends and bank holidays where flow breakdown is infrequent, hence the total days that are, in principle, available is reduced to 170.

Among these remaining 170 days, parts of them are affected by incidents or MIDAS system failure. After excluding these data, the numbers of days available for entry and exit daily demand analysis are listed in Table 7.1 and Table 7.2 respectively.

Table 7.1 Data Availability – Number of Days for Entry Daily Demand Analysis

		Entry				Tot(available for four entries)
		M6	M5	J9	J10	
Incident-free days	Monday	13	13	11	13	11
	Tuesday	22	22	19	23	18
	Wednesday	20	20	15	20	15
	Thursday	20	20	17	20	17
	Friday	18	18	15	18	15
	Total	93	93	77	94	76

Table 7.2 Data Availability – Number of Days for Exit Daily Demand Analysis

		Exit		Tot(available for two exits)
		J9	J10	
Incident-free days	Monday	13	13	13
	Tuesday	23	22	22
	Wednesday	20	19	19
	Thursday	20	20	20
	Friday	18	17	17
	Total	94	91	91

The numbers in the "Tot" column in the above two tables are the days where the data is available for **all** entries or exits, and must be equal to or less than the minimum number of days available for the individual entries/exits (since the missing days may not be the same across all the entries or exits).

Note that strictly a flow figure can only be regarded as demand when there is no queueing at the entry point (or exit, as off slip queues can occur due to junction congestion). This is particularly important for the analysis of profiles, since there will be a direct effect on the time at which the demand enters the system, relative to the desired time. It is less important, however, for the analysis of total daily demand, since, apart from traffic which either diverts or is suppressed, the demand will – sooner or later – be translated into flow. This means that the treatment of data needs to be different.

For the analysis of the total daily demand, we only need to patch for the missing 5min intervals. The method adopted is described below.

The data has the form x_t^{iy}, but for any particular day y, the information for some of the intervals t may be missing. Denote this by the binary variable δ_t^{iy} which has the value 1 if the data is present, and 0 if missing.

We first construct the overall average $\bar{x}_t^i$ for each interval t, as follows:

$$\overline{x}_t^i = \frac{\sum_y \delta_t^{iy}.x_t^{iy}}{\sum_y \delta_t^{iy}}$$

where the denominator represents the number of days for which data was available. We then use this to give us the average total daily demand $\overline{x}_*^i$ given by

$$\overline{x}_*^i = \sum_t \overline{x}_t^i$$

Now consider an individual day y for which data is missing in some intervals t. To calculate $\hat{x}_*^{iy}$ the total daily demand for day y, we use the following formula:

$$\hat{x}_*^{iy} = \left(\sum_t \delta_t^{iy}.x_t^{iy}\right) \frac{\overline{x}_*^i}{\sum_t \delta_t^{iy}.\overline{x}_t^i}$$

(in other words, we factor up the total for the available intervals according to the ratio of the average total daily demand to the average total demand for the same set of intervals).

For the analysis of the 5min demand profile, the number of days has been further reduced due to stricter requirements relating to the level of missing data, as well as the treatment of time periods where flow is not considered to represent demand. This is discussed later.

7.3 General Method of Analysis

The general approach is to use Analysis of Variance [ANOVA] techniques to distinguish between different effects. Since the testing for the total daily demand is relatively straightforward, we will begin with a description for this, and then discuss the profiles.

For the total daily demand, we have separate estimates $\hat{x}_*^{iy}$ for each day y and each entry i. In this case, the variation by entry is clearly required input, and there is no need to carry out tests as to whether the values are significantly different. We therefore confine the description to the treatment of days.

In principle, the individual days could be grouped into categories {Y} (for example, all y falling on Friday could be grouped), and we could test whether the grouping accounted for a significant "explanation" of the variation. This is/would be done along the following lines:

- Calculate overall mean: $\overline{x}_*^i$

- Calculate means for each group $\overline{x}_Y^i = \frac{\sum_{y \in Y} \overline{x}_y^i}{N_Y}$ where N_Y is the number of days in group Y

- Set out the sum of squared deviations according to the following table:

	SS	df	Mean SS= SS/df
total	$\sum_y \left(\overline{x}_y^i - \overline{x}_*^i\right)^2$	no. of days – 1	A
group effect	$\sum_Y N_Y \left(\overline{x}_Y^i - \overline{x}_*^i\right)^2$	no. of groups – 1	B

residual	$\sum_{Y}\sum_{y\in Y}\left(\bar{x}_y^i - \bar{x}_Y^i\right)^2$	no. of days − no. of groups	C

The ratio B/C is distributed as an F-variate with degrees of freedom equal to (no. of groups − 1, no. of days − no. of groups). If the value is associated with a low probability (eg less than 0.05), it may be concluded that the grouping represents a significant effect.

A further test is then appropriate to verify whether the categories within the grouping are all significantly different. There are essentially two different ways in which this can be done, though the conclusions should be the same. A straightforward way is to test all pairs of categories, carrying out a standard t-test. In doing so, it is advisable to introduce a correction for multiple comparisons. Thus the test may be described as follows:

- Calculate difference between group means Y and Z: $\Delta = \bar{x}_Y^i - \bar{x}_Z^i$

- Calculate variance of difference assuming independence: $\Sigma^2 \approx \mathrm{var}(\bar{x}_Y^i) + \mathrm{var}(\bar{x}_Z^i)$

- Compare Δ/Σ with a t-statistic: if it exceeds the critical value, the difference between the means may be considered significant

If the standard t-statistic is used, this is appropriate for a single pair-wise comparison. The effect of the correction for multiple comparisons is to inflate the t-statistic so that it is less likely that one of the comparisons will be considered significant by chance.

The alternative approach is to test a reduced grouping (eg by conflating two of the groups Y and Z into a single group W) and then to carry out an ANOVA procedure for the difference in explanation. Writing Y as the generic group in the prior case, and Y′ as the generic group in the reduced case, where the groups in Y′ "nest" within the groups in Y (ie, no overlapping), the following procedure may be applied:

Set out the sum of squared deviations according to the following table:

	SS	df	Mean SS= SS/df
residual Y	$\sum_{Y}\sum_{y\in Y}\left(\bar{x}_y^i - \bar{x}_Y^i\right)^2$	no. of days − no. of Y groups	A
residual Y′	$\sum_{Y}\sum_{y\in Y}\left(\bar{x}_y^i - \bar{x}_Y^i\right)^2$	no. of days − no. of Y′groups	B
difference			C

The ratio C/A is distributed as an F-variate with degrees of freedom equal to (difference in no. of groups, no. of days − no. of Y groups). If the value is associated with a low probability (eg less than 0.05), it may be concluded that the reduced grouping represents a significant effect – in other words, it cannot be justified.

Both these approaches have been used in the subsequent analysis.

In the case of the profiles, we are comparing **vectors** of means rather than single quantities, and in principle this could introduce significant complexity. We have adopted a simpler ANOVA

approach which simply sums the squared deviations over all time intervals without taking account of the pattern of deviations (eg to check for serial correlation).

For the profiles, in principle we would wish to calculate the proportion of the estimated total demand $\hat{x}_*^{iy}$ which falls in each time interval t – in other words $x_t^{iy} / \hat{x}_*^{iy}$. In practice, for reasons of missing data, we have chosen to calculate the profiles as a proportion of the average demand over a period of the data which is least susceptible to missing data problems. Here we ignore details and merely outline the general method of analysis.

Assume that we notate the profiles as R_t^{iy}, defined for each day y, each entry i, and each time interval t. The aim is to determine whether the profiles vary by particular groupings of days (as was done for the total daily demand above), **and** by entries. As before, we confine the general description to the treatment of days.

To test a particular grouping {Y}, we:

- calculate overall mean profile: $$\overline{R}_t^{i} = \frac{\sum_y R_t^{iy}}{N_t^i}$$ where N_t^i is the number of days with observations for period t for entry i

- calculate mean profiles for each group: $$\overline{R}_t^{iY} = \frac{\sum_{y \in Y} R_t^{iy}}{N_t^{iY}}$$ where N_Y is the number of days in group Y with observations for period t for entry i

- set out the sum of squared deviations according to the following table:

	SS	df	Mean SS= SS/df
total	$\sum_t \sum_y \left(R_t^{iy} - \overline{R}_t^{i} \right)^2$	no. of days – 1	A
group effect	$\sum_t \sum_Y N^{iY} \left(\overline{R}_t^{iY} - \overline{R}_t^{i} \right)^2$	no. of groups – 1	B
residual	$\sum_t \sum_Y \sum_{y \in Y} \left(R_t^{iy} - \overline{R}_t^{iY} \right)^2$	no. of days – no. of groups	C

As before, the ratio B/C is distributed as an F-variate with degrees of freedom equal to (no. of groups – 1, no. of days – no. of groups). If the value is associated with a low probability (eg less than 0.05), it may be concluded that the grouping represents a significant effect. By analogy, a similar ANOVA approach is then appropriate to verify whether the categories within the grouping are all significantly different.

7.4   Results for Variation in Total Daily Demand

The analysis is reported in detail in Hyder (June 2010). Here we merely give the recommendations for the treatment of average daily demand, shown in Table 7.3:

Table 7.3: Day of Week Groupings for Average Daily Demand

Entry	No. of groups	Group	Mean	SD	CV
M6	2	M-T-W-F	46360	1999	0.04
		Thu	47686	1449	0.03
M5	1	M-T-W-T-F	28896	967	0.03
J9	2	M-T-W-T	5751	527	0.09
		Fri	6661	568	0.09
J10	5	Mon	8946	407	0.05
		Tue	9445	608	0.06
		Wed	9734	527	0.05
		Thu	10053	637	0.06
		Fri	10728	522	0.05

These conclusions were based on the analysis of the data summarised in Table 7.1, with the patching for missing periods described in the previous section. In terms of mean daily demand, the difference between term time and school holidays was not found to be significant, though various effects relating to the day of week were found, as summarised in the above table. In addition, correlations should be allowed for between the J9 and J10 entries, of the level of 0.63 on Fridays and 0.57 on other days. This implies that the distribution for these two entries should be taken as jointly normal. Further, on Fridays a negative correlation of –0.52 is found between the M5 and J9. All other entries on all other days can be assumed uncorrelated. The underlying analysis suggested that correlations between entries and exits are generally likely to be highly location-specific depending on the areas served by the routes and the nature of the network (e.g. the potential for switching between routes).

Following this initial exercise a further analysis of the data was undertaken of the MIDAS data. This identified a number of days during which 5minute interval readings were clearly faulty (mainly zero readings). These days were excluded from the analysis and as a result the figures for variation in total daily demand, shown in Table 7.4, are used in the Aggregate Model.

Table 7.4: Variation in Total Daily Demand – Coefficient of Variation

Entry CV	Monday	Tuesday	Wednesday	Thursday	Friday
M6	0.023	0.046	0.024	0.026	0.023
M5	0.022	0.036	0.022	0.022	0.025
J9	0.025	0.093	0.062	0.091	0.082
J10	0.042	0.052	0.044	0.039	0.043

Exit CV	Monday	Tuesday	Wednesday	Thursday	Friday
J9d	0.038	0.049	0.042	0.06	0.055
J10d	0.063	0.061	0.034	0.066	0.068

7.5   Analysis of Daily Profile

The aim in this case was to analyse and produce a model of the daily demand profile (mean and variance) for all four entries (M6, M5, J9 on-slip and J10 on-slip), and the two exits, J9 offslip and J10 offslip. Because many of the days had some missing 5min flows in the MIDAS database, this caused some complexity.

As a starting point of the demand analysis, the mean MIDAS flow counts have been plotted in Figures 7.1 to 7.4 in order to obtain an initial idea about the flow profiles for each day of week at each entry. The mean profiles come from the data described in Table 7.1, with missing 5min intervals excluded.

The analysis is generally based on the data described in Table 7.1. However, because the method we are using to estimate demand profile during flow breakdown periods requires the definitions of flow breakdowns starting time and end time for each day (provided by the BDF and QDF analysis – Section 8), this further limits the selection of data, as follows:

Table 7.5 Data availability for the analysis of profiles

	M6	M5	J9	J10
Monday	8	8	5	8
Tuesday	17	17	13	17
Wednesday	15	15	10	15
Thursday	14	14	9	14
Friday	13	13	9	12
Total	67	67	46	66

Figure 7.1 M6 average entry flow profile for each day of the week (all vehicles)
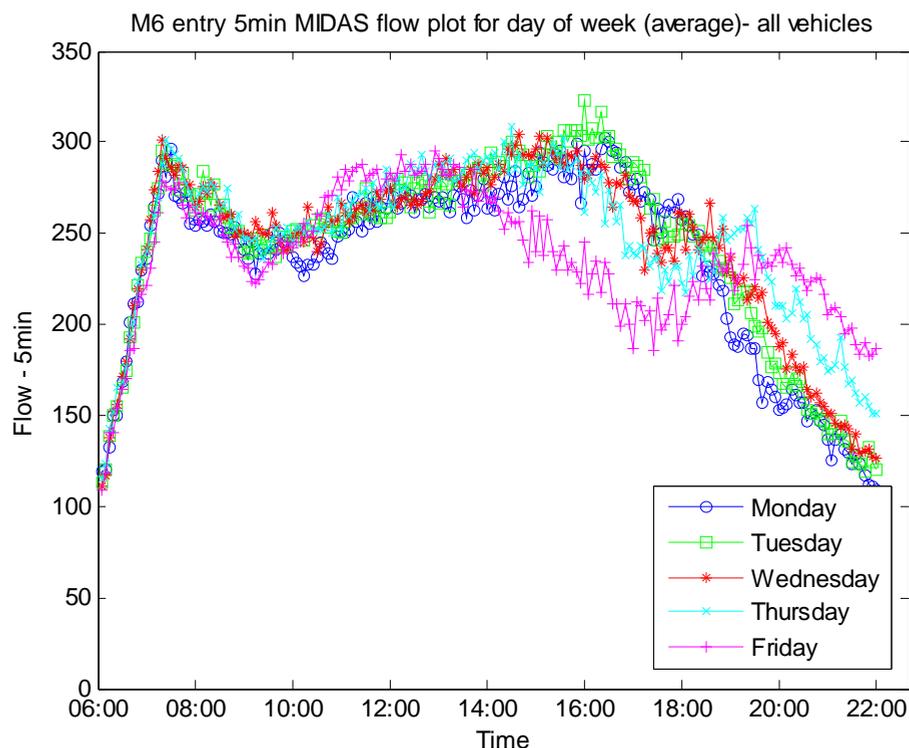
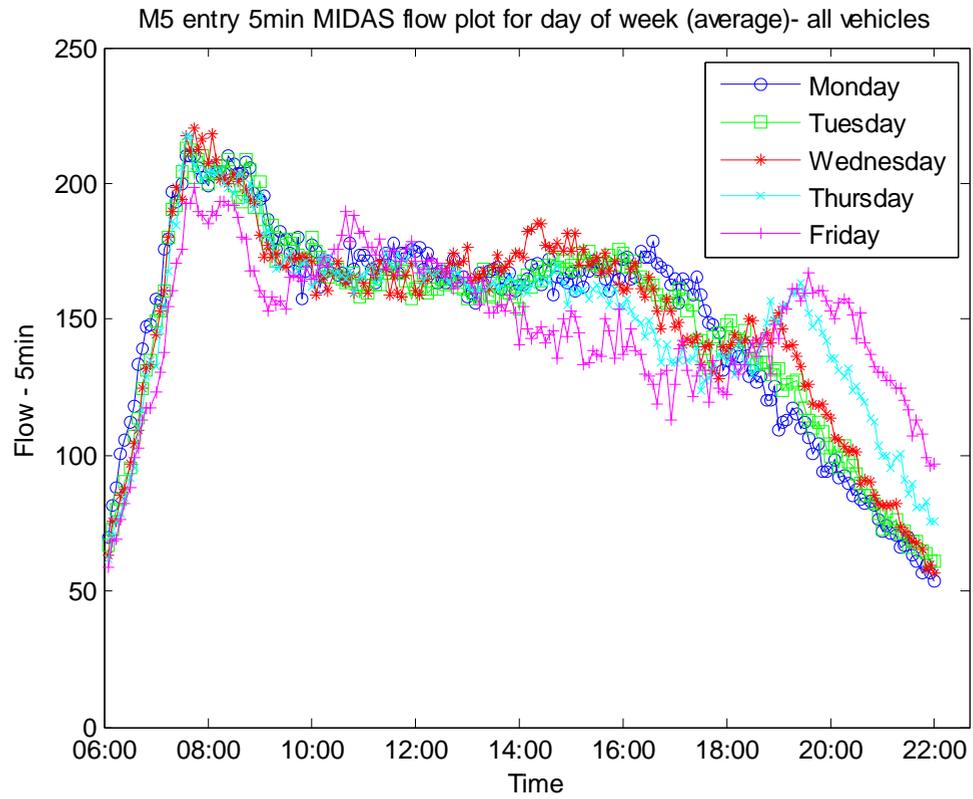**Figure 7.2: M5 average entry flow profile for each day of the week (all vehicles)**

M5 entry 5min MIDAS flow plot for day of week (average)- all vehicles

Figure 7.3: J9 average entry flow profile for each day of the week (all vehicles)

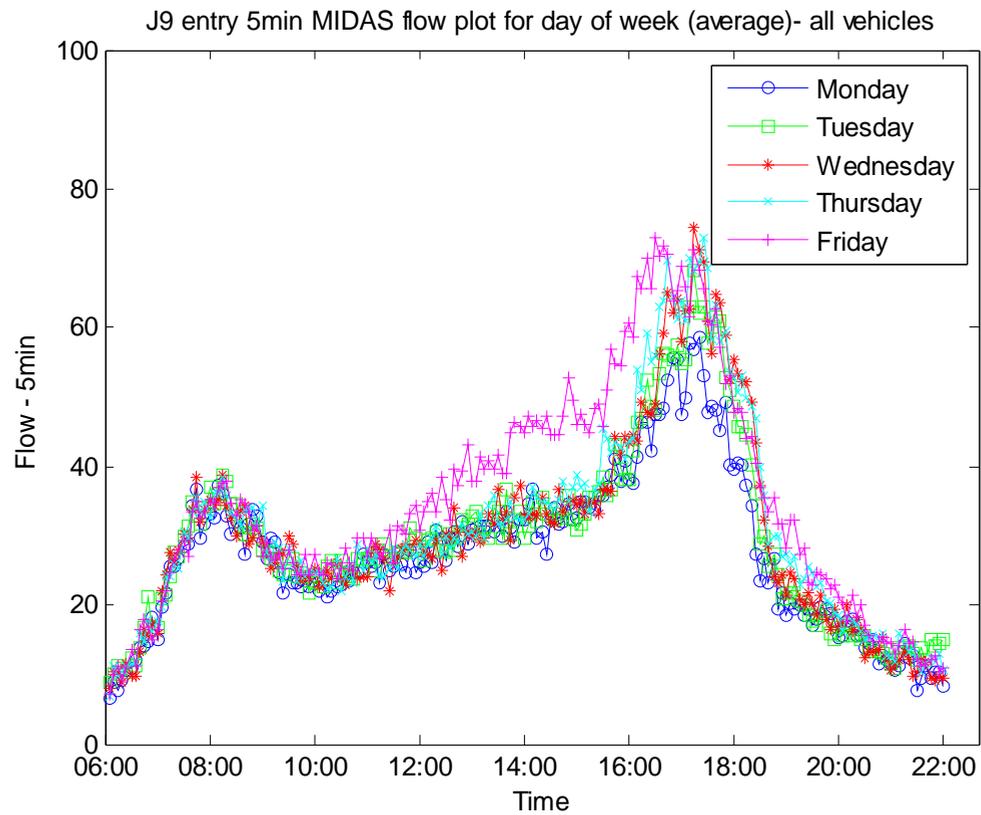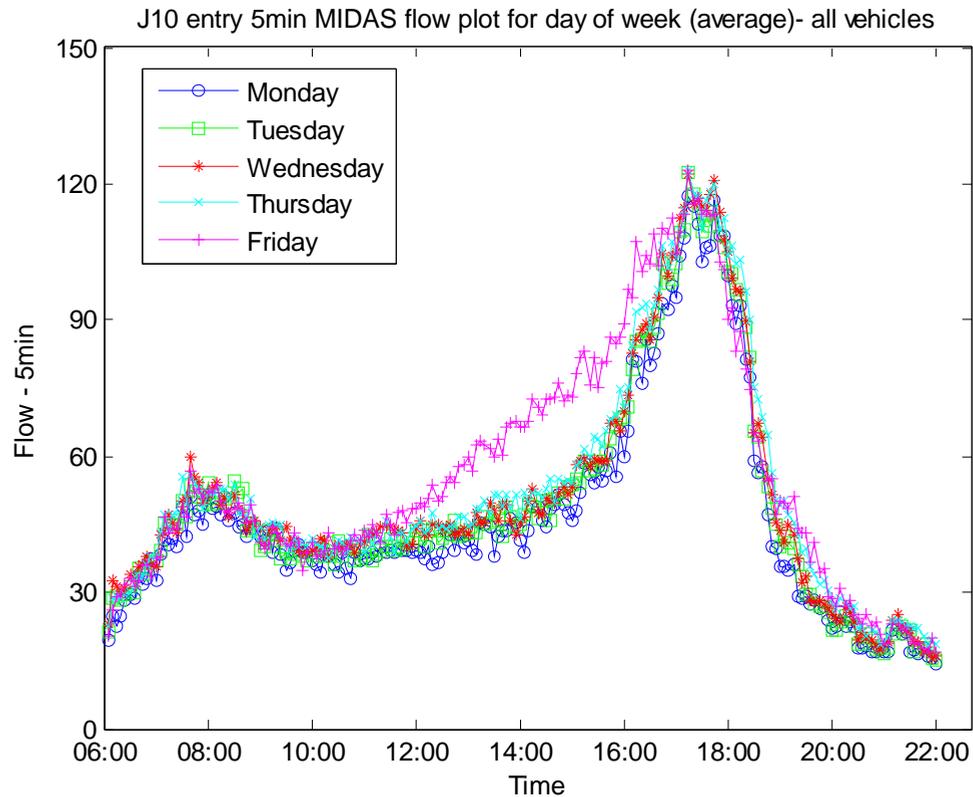J9 entry 5min MIDAS flow plot for day of week (average)- all vehicles

Figure 7.4: J10 average entry flow profile for each day of the week (all vehicles)

From Figures 7.1 to 7.4, three different patterns can be identified. Apart from Fridays, flows on the mainline M5 and M6 entries show a similar pattern between 06:00 and 16:30, before flow breakdown occurs. However, after that, flow falls consistently from the time flow breakdown occurs (around 16:30) until around 18:00 after which it rises again. On Mondays, where there is little flow breakdown at J8m the demand is fairly constant throughout the afternoon. Fridays show a different picture, with flow higher than other weekdays from 10:30 until 13:00 before breakdown occurs. Then the flow falls further than any other days.

For periods where there is no flow breakdown the MIDAS counts are considered to give the true demand i.e. vehicles wishing to enter the system at that time. Once flow breaks down and queues occur they give the flow in the queue at their location. As we should have some idea of the queue length at any point in time from the MTV plots, we could make an estimate of the actual demand in any time slot by making some assumptions on headways between vehicles (this could be done in 5 minute periods). The complication is that, once the queue extends beyond the M6 J8 diverge, the queue will also include traffic going to M5 south: thus the demand joining the queue is much higher than that for the northbound J8 merge. If the data on all the flows in the J8 area were available it might be possible to estimate the actual demand for J8m, but unfortunately this data is unavailable for this study. Also due to long term roadworks there are no MTTV plots available to identify the end of the queue when it extends beyond J7. As a result we do not really consider it possible to estimate the true demand for J8m.

On the two on-slip entries, all days have a similar profile except for Fridays which have higher flow from 12:00 until 17:00. It is possible that this is caused by the significant diversion from M5 J1 to M6 J10, as well as possible diversion from M6 J7 to J9 to avoid the queue at the J8 merge. In consequence some of the apparent demand at these merges may in reality be demand diverted from the M5/M6 entries.

Note that the peak for the J9 on-slip and J10 on-slip flow is around 17:30 – 18:00, which is the lowest flow point of the M6 and M5 entries. After 18:00, the flow from J9 and J10 on-slips starts to decline rapidly, while both the M6 and M5 flow begin to increase again. This suggests that the flow from the on-slip roads can severely affect traffic flow on the mainline sections.

After solving the missing data problems, another issue, as discussed in section 7.1, is how to estimate traffic demand during flow breakdown periods. Although it is not possible to obtain the true demands, especially when traffic queues beyond the M6 J8 diverge or the M5 J8 diverge, an approximate demand profile shape can still be obtained based on ANPR travel time data. In general, travel time tends to increase while the queue is building up (more entry demands than merge capacity, i.e. QDF, Queue Discharge Flow) and reduce while it is declining (QDF exceeds entry demand). The methodology is explained in Figure 7.5:

Figure 7.5 Method to estimate demand during flow breakdown (illustration for 12/02/2003)
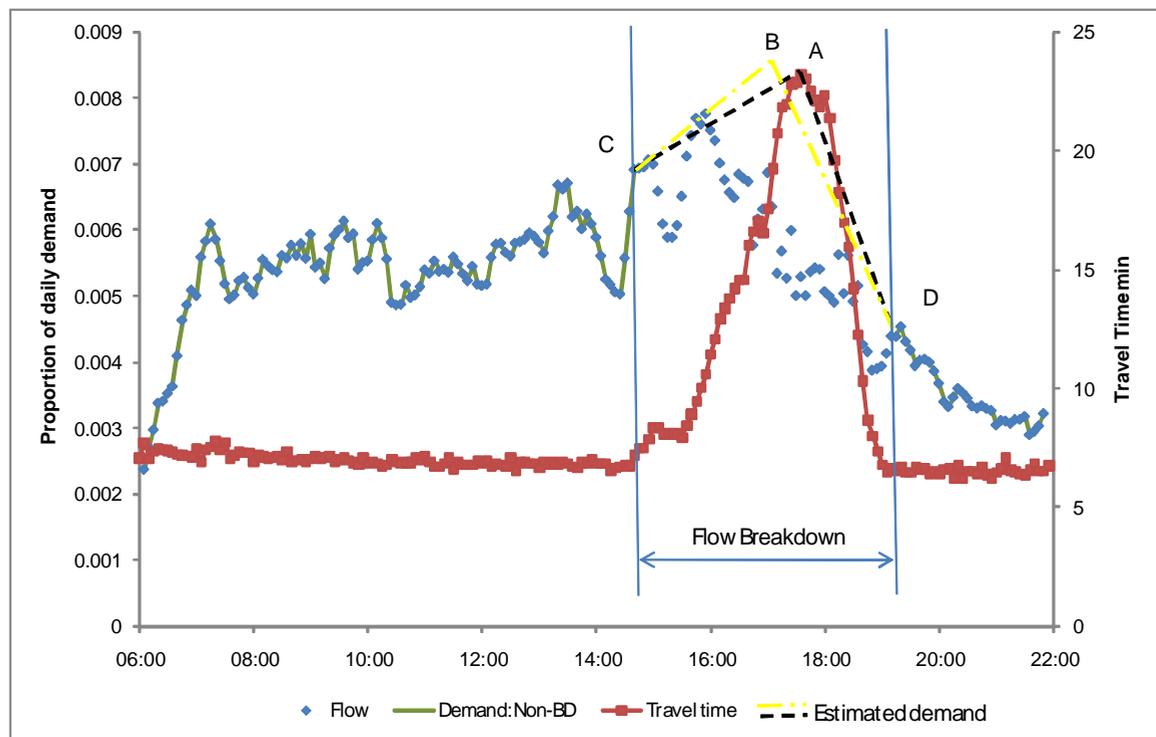
The blue diamonds in Figure 7.5 show the 15min moving average proportion data for one particular day 12/02/2003 (5min MIDAS flow data for the M6 entry divided by total daily demand) and the red line shows the corresponding ANPR travel time data (cameras 1a-4). The flow breakdown start time and end time for this day (14:15 and 19:00 respectively) have separately been identified during the BDF and QDF analysis (see Section 8). In the above figure, the green line represents traffic demand, which is assumed to be the same as flow outside the flow breakdown period.

With the help of the ANPR data, the peak point (A) can be identified. This peak point A is assumed to be the last 5min interval where the entry demands exceeded merge capacity. Subsequent to this point, travel time started to decline and this implies that the entry demands in the following intervals were less than the merge capacity. By connecting point A with the last point of pre-BD profile (point C) and the first point of post-BD profile (point D), an estimate of the **shape** (but not the height) of true demand is obtained, shown as the dashed black line.

Having identified the true demand shape, the only problem remaining is to estimate the proportion value (vertical value) of this peak point (A). Since $\mathbf{p}^{if}$ is the proportion profile vector, the sum value of $\mathbf{p}^{if}$ should always equal to 1. Given that the values of non-BD periods demand are known, together with the horizontal location of point A, the vertical value of A can be easily calculated. Hence, the true demand profile for the 12/02/2003 is estimated. Although this has not been shown in the figure, the approach has always ensured that the overall demand during BD is correctly reproduced.

Although this method can give an approximate estimate of the true demand profile, there are some drawbacks:

- The maximum travel time between cameras 1a and 4 will probably not correspond with the highest demand, as the size of queue will continue to increase as long as entry demands exceed merge capacity. In addition, some account should be taken of where the cameras are located, especially when the queue extends upstream of them. A more critical factor in increasing JTs is the reduction in QDF due to blocking back from J10/J9. The maximum demand is when the rate of increase of the queue length is at its maximum, which is shown as point B in Figure 7.5. As the yellow line shows, this will cause slight changes to the estimated demand shape, but the differences are small.

- If the queue extends beyond camera 1a, ANPR data will not give us the accurate information about the highest demand. However, from the point of view of the Aggregate Model, which will be calibrated and validated against ANPR data, this probably is not important.

- In an ideal situation, it would be preferable to use cameras 1a-2 for estimating the M6 entry demand, cameras 1b-2 for estimating the M5 entry demand, and cameras 2-4 for the J9 and J10 entries demand. However, for the sake of simplicity and given the quality of ANPR data (due to a low sample rate on some lanes in the ANPR data for those pairs), only 1a-4 has been used here. This could be extended in the future to obtain a potentially more accurate demand profile.

In the course of testing the model, it became apparent that this "fixed" profile could not generate the observed breakdown and delay profile. On further consideration we concluded that the profile should not suddenly start to decline at the beginning of the breakdown period. A new profile was therefore developed in which demand continued to increase before eventually declining. However, in order to maintain the total demand unchanged, a period of constant demand is generated near the end of the flow breakdown period, (as will be seen in Figure 7.6 below). Although this revised profile is still unlikely to be the true shape of demand profile, it looks more sensible than the former profile, especially for the early breakdown periods.

7.6   Results for Average Daily Profile

Similar to the daily demand analysis, the analysis of variance (ANOVA) was carried out to test for differences by day of week. Once again, the details are reported in Hyder (June 2010), and here we merely summarise the final conclusions.

The ANOVA analysis suggests that apart from the J9 entry, the day of week factor is statistically significant – in other words, the temporal pattern of entries generally varies by day of week – in contrast to the conclusions for total daily demand. After investigating appropriate groupings, the results are as in Table 7.6:

Table 7.6: Groupings for the Day of Week at each Entry

	Number of Groups	Group			
		1	2	3	4
M6	4	Mon	Tue-Wed	Thu	Fri
M5	4	Mon	Tue-Wed	Thu	Fri
J9	1	M-T-W-T-F			
J10	2	M-T-W-T	Fri		

ANOVA analysis was also used to see if the profiles varied by entry. As a starting point, the adjusted percentage profiles for each day of the week are plotted to see if any of the entries can be combined. It was concluded that the temporal pattern of demand was significantly different for all entries.

Hence, it was concluded that a total of 11 distinct profiles, for the combination of weekdays at four entries, as indicated in Table 7.6, needed to be generated for the Aggregate Model. The resulting profiles for the mean have been plotted in Figure 7.6. For non-breakdown periods, the demand proportions are equal to observed proportions; for breakdown periods, the demand proportions are estimated using the method described in Section 7.5. The use of a period of constant demand near the end of the flow breakdown period (as described in the final paragraph of Section 7.5) explains the straight line segment on the mean demand profile for Friday at the M6 entry.

Figure 7.6: Mean Demand Profiles for the four Entry Points
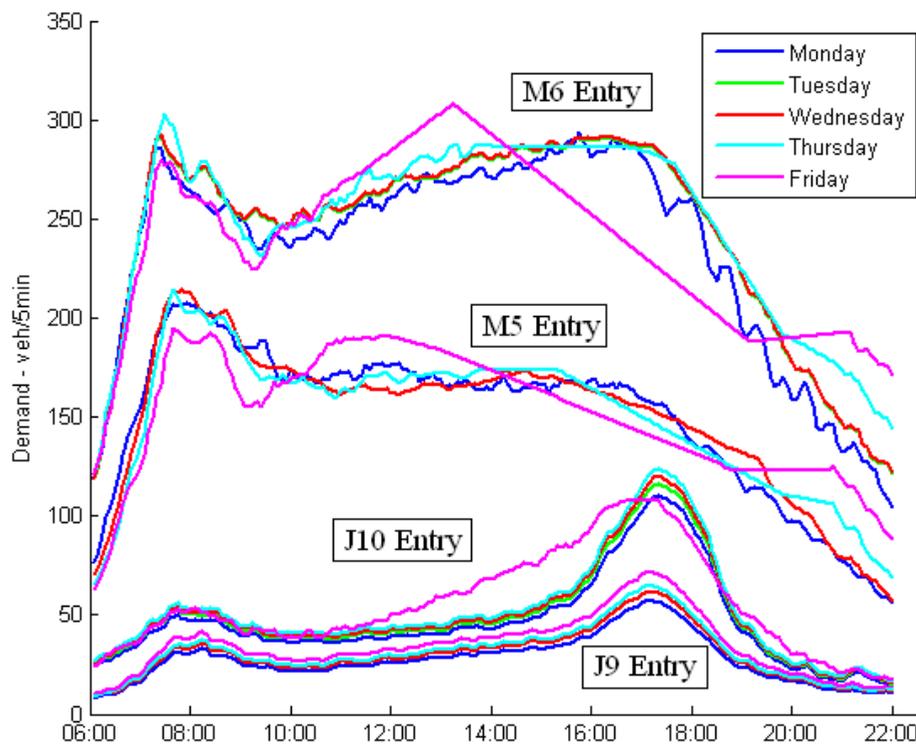
A similar analysis for the two exits revealed the mean demand profiles shown in Figure 7.7:

Figure 7.7: Mean Demand Profiles for the Two Exit Points
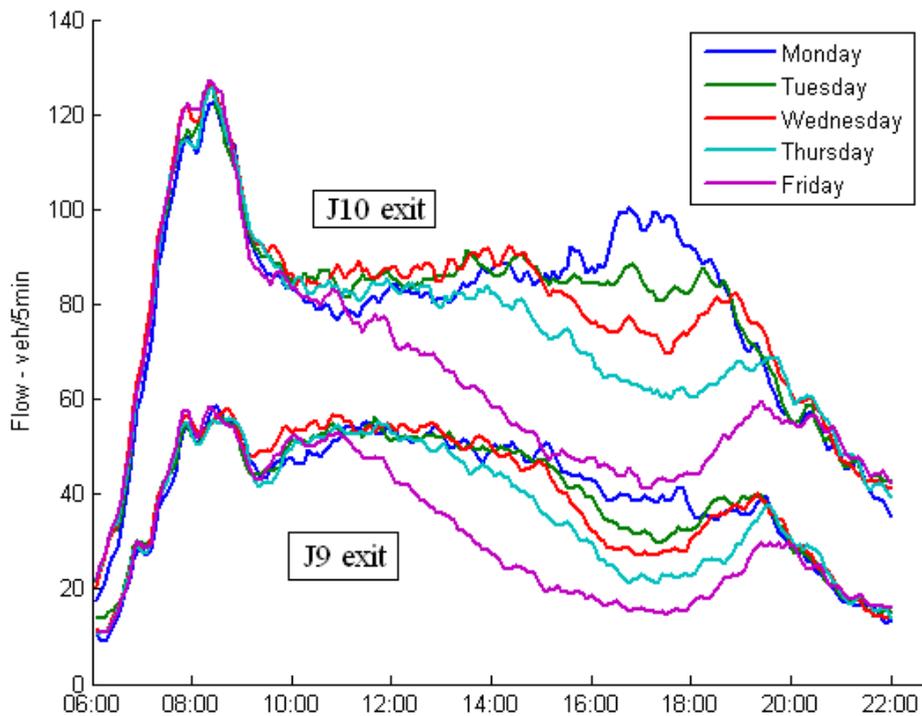
While for M6 and M5 the Thursday and Friday profiles look low for the flow breakdown period, this is due to the higher proportions of the post-breakdown periods for Thursday and Friday, and needs to be seen in the light of the total daily demand.(sum of the four entries), for which Thursday is the highest, followed by Friday, Wednesday, Thursday and Monday. Hence although the Thursday and Friday proportions are lower during the BD periods, the absolute numbers of vehicles are still generally higher.

7.7 The Variation in 5 minute Profile Intervals

Figures 7.6 and 7.7 are mean profiles. In practice we find considerable fluctuation around these mean profiles. i.e. there is stochastic variation around the profile in each 5 min interval. The coefficient of variation (CV) of the profiles can be calculated in two ways.

The first method calculates the CV based on all valid observations of the profile for each 5 minute interval, and then takes the average over all 5 minute intervals. The results of this exercise are shown in Table 7.7 for the entries and in Table 7.8 for the exits.

**Table 7.7 Average CV for the four Entry Points by Day of Week Grouping**

	No. of Groups	Average 5min CV for each group			
		1	2	3	4
M6	4	0.065799	0.072304	0.063489	0.077641
M5	4	0.077114	0.090408	0.074593	0.078852
J9	1	0.171379			
J10	2	0.115862	0.122988		

Table 7.8: CV for the two Exit Points by Day of Week

|	J9	J10
Monday	0.132	0.100
Tuesday	0.159	0.114
Wednesday	0.154	0.105
Thursday	0.191	0.136
Friday	0.182	0.136

An alternative approach to estimating the 5minute CV is to consider it as a temporal phenomenon. A single daily demand profile for an entry is generated from the average demand profile of that entry and two sources of variability. The first is day to day variability. For this variability the average demand profile is scaled by a stochastic factor. The second source of variability is concerned with how the demand distributes itself within 5 minute intervals. Vehicles arriving at junction 8 on the M6 will not exhibit a smooth profile – some 5 minute intervals will be higher than the profile and others lower. Figure 7.8 illustrates the pattern of arrivals for a day on the J9 entry slip. The vertical axis represents the ratio of demand to the average demand profile. Missing values due to no data or breakdown conditions are shown as blank.

Figure 7.8 Fluctuation in flow around the mean profile on one day - J9 entry
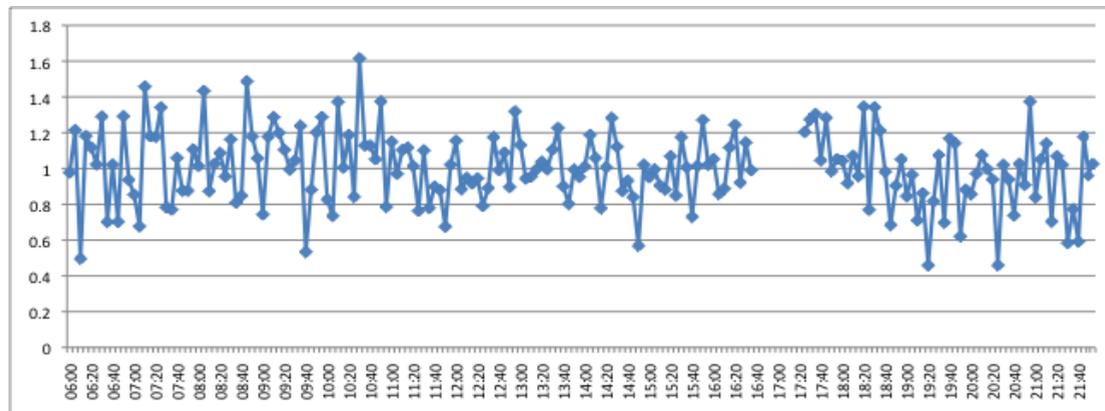
An estimate of the CV is straightforward in this instance. However for other days (such as the one in Figure 7.9) the deviation from average profile contains trends that are not relevant to the 5 min variation we are interested in. One way of solving this problem is to fit polynomial curves to the data in order to eliminate the within day trends that occur when the particular day's profile is not a simple factor of the average day profile.

**Figure 7.9   Ratio of Demand to Average Profile Demand – 13/08/03 M5 entry**
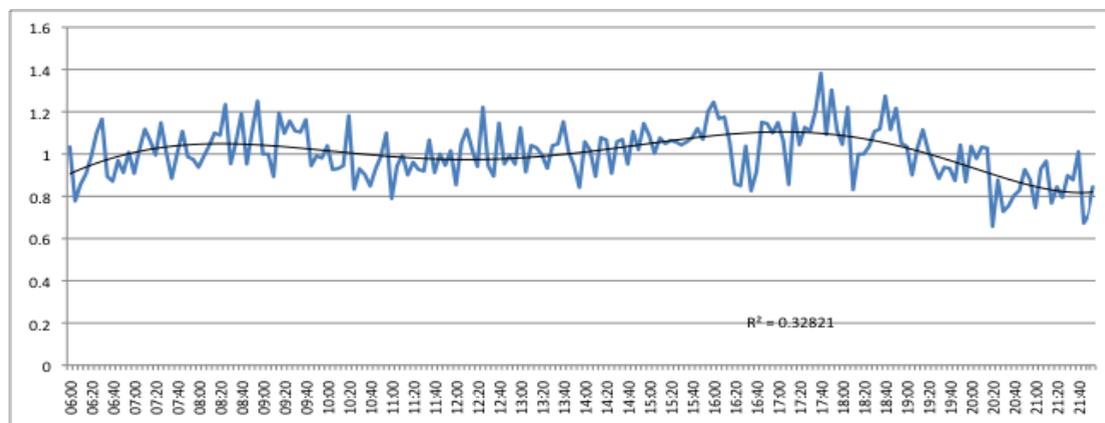
In the case shown by fitting a polynomial curve the standard deviation is reduced from 0.124 to 0.102. Applying the polynomial fitting to a set of days (in practice only part of the day) reveals the following results. This fluctuation in 5 minute demand is a traffic phenomenon and therefore should not vary by day of week. The data confirms there is no significant difference in days of week.

Table 7.9   Estimates of 5min CV and its correlation

Entry	M6	M5	J9 ramp	J10 ramp
5min CV	0.09	0.09	0.21	0.16
autocorrelation	0.4	0.4	<0.1	<0.1

An assumption in the Aggregate Model is that the 5min variation is Normally distributed. As can be seen in Figure 7.10, for the M6 observations this is broadly true, although there is a slight tendency for observations to concentrate near 1.0. The upper tail (which is critical in generating breakdown) using this figure shows no serious deviation from Normality. A similar pattern is found for other entries.

Figure 7.10 Distribution of Demand to Average Profile Demand – M6 Entry
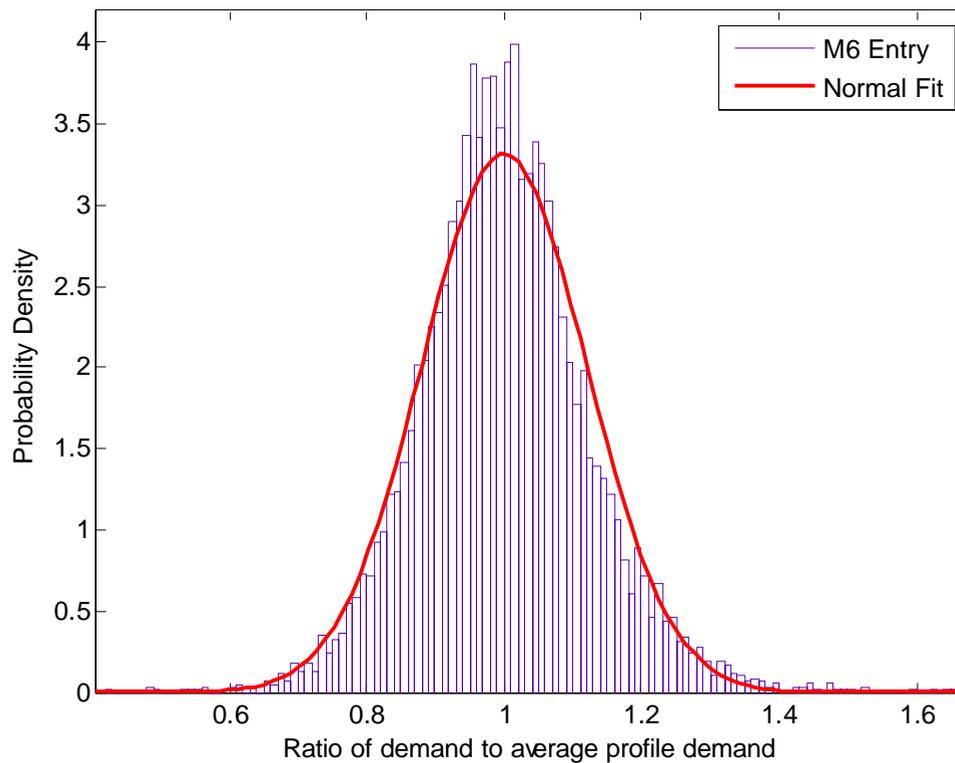
7.8   Conclusions

Considering that the data was collected over a 49-week period, the actual amount of usable data for the purpose of the analysis turns out to be quite limited. To some extent, this is a

function of the aims of the study, since we have excluded weekends on the grounds that flow breakdown (other than that caused by incidents) is generally not present. For similar reasons, we have to exclude periods when traffic is disrupted by incidents. For the remaining periods of interest, there are further problems of data to contend with, particularly those related to equipment failure or non-availability. Finally, there is the key issue of how far the observed MIDAS flows can be taken as representative of demand.

We chose to divide the general issue of demand variability into two factors – the total daily demand, and the proportionate allocation ("profile") of that demand across the day.

Based on the analysis reported here, it is clear that there are important differences in:

- the mean daily profile between all entries;
- the mean daily profile between days of week (especially Friday);
- the total daily demand variation (CV) between the two motorway and two slip entries; and
- the 5minute variation between the two motorway and two slip entries.

There are only minor differences between the total daily demand variation by day of week.

The input data for the Aggregate Model corresponds to:

- mean daily profiles by day of week and entry/exit – Figures 7.6, 7.7
- total daily demand variation by day of week and entry/exit (CV)– Table 7.4
- five minute interval variation (CV) by entry – Table 7.9

8 FLOW BREAKDOWN ANALYSIS

8.1 Introduction

In this Section we describe the analysis for the BDF (Breakdown Flow) and QDF (Queue Discharge Flow) functions at the merges at M6 junctions 8, 9 and 10.

The general approach to analysing each individual day (given that is was suitable) was as follows:

- MTV plots were used to determine the approximate times of breakdown at the three individual junctions;

- Speed plots for the MIDAS loops in proximity to each junction were then produced for the applicable times of the day, as determined by the MTV plots, from which a more precise time and location of breakdown could be determined;

- The 5-minute intervals for each junction were then tagged if necessary as being when breakdown was initiated or during the following breakdown period; and

- The MTV plots were also used to determine when blocking back from downstream junctions affected the QDF and these periods were given a separate tag.

For the BDF calculations, flows from upstream loops on the M6 mainline and on-slip were used. This gives a better indication of the arrival flow which causes the breakdown in flow. For the QDF calculations, loops from downstream of the merge were used. This gives a direct indication of capacity by measuring the outflow from the head of the standing wave at the merge. The loops used for each junction are shown in Table 8.1:

Table 8.1 Loops used for BDF/QDF Calculations

BDF Calculations	
Junction	MIDAS Loops Used
10	5979A, 5984K
9	5955A, 5959K
8	5920A, 7003B
QDF Calculations	
Junction	MIDAS Loops Used
10	6000A
9	5969A
8	5951A

The outputs generated a probability distribution for breakdown related to flow at each junction, an average 5-minute QDF for each individual junction and also an average 5-minute QDF whilst blocking back from downstream was apparent for each junction.

While the work was originally confined to the 25 days with no incidents and minimal missing data (see §3.5), it was decided that more data was required to derive robust conclusions, and a further 56 days were identified that could be analysed, thus making 81 in total. A summary of the days used is shown in Tables 8.2 and 8.3.

Table 8.2 Days Analysed by Month

Month	Days	Percentage
January	1	1.23%
February	9	11.11%
March	5	6.17%
April	12	14.81%
May	10	12.35%
June	0	0.00%
July	6	7.41%
August	11	13.58%
September	13	16.05%
October	11	13.58%
November	3	3.70%
December	0	0.00%

Table 8.3 Days Analysed by Day

Month	Days	Percentage
Monday	11	13.58%
Tuesday	20	24.69%
Wednesday	17	20.99%
Thursday	18	22.22%
Friday	15	18.52%

This shows that the observed data is sourced from a variety of months (there is no June data due to a MIDAS system failure), and is distributed relatively evenly amongst each weekday. Monday was the day analysed the least, but generally they did show the fewest occurrences of breakdown of all the days.

8.2 The Analysis

It was decided that the onset of breakdown should not be dictated by a drop below any particular speed, especially not one as high as 80 km/h (see the citation in §1.3). It was agreed that an approach based both on speed and the change in speed should be used. No fixed criteria were set, a certain level of flexibility would be allowed to the analyst to determine exactly when breakdown had occurred. Typically if speeds fell at a significant rate to below 60 km/h, this was tagged as being the start of breakdown.

The 5-minute period where breakdown was deemed to have begun was based on the speed plots for each of the loops in proximity to the merges. Subsequent analysis of minute-by-minute speeds confirmed this period or allowed slight modifications to be made.

In terms of deciding when flow breakdown has ceased, an increase in speed back above approximately 60-70km/h for a reasonable length of time, on the loop adjacent to the given merge, was deemed to be an appropriate indication.

In addition, precise 1-minute breakdown times and flows were determined by the following method:

- Using the location and approximate time of the breakdown, loops speeds were analysed to find the exact minute when breakdown was deemed to have commenced; and

- 1-minute flows were then taken from the upstream counts (mid-junction and on-slip loops) taking into account the approximate time it would take vehicles to reach the breakdown location.

This resulted in 1-minute flows being produced for each instance of flow breakdown on each day. It was noted that that this had a significant margin of error, as one analyst's interpretation of the exact minute of breakdown may differ from another. Also the assumptions for time taken to reach the breakdown location from the upstream loops could only be made to the nearest minute, so could also lead to potential errors.

The first step in the analysis was to scan each 5-minute interval for which data was available between 0600 and 2200, and to judge whether or not it represented the initiation of flow breakdown, separately for each junction. If it was so judged, then the duration of the subsequent breakdown period needed to be determined. In this way, separately for each junction, each available 5-minute interval could be classified into one of the three following categories:

- no breakdown

- breakdown initiation

- within duration of breakdown periods

For the total of 15552 five-minute periods for the 81 days analysed, breakdown occurrences are summarised in Table 8.4. This shows that flow breakdown occurs most frequently, on average 1.7 times per day, at J8m and least frequently at J9m. The flow breakdown is J9m is mainly associated with block backs from J10m.

Table 8.4 Flow Breakdown Status by 5 Minute Period

Merge	FB Initiation	FB No Blockback	FB Blockback
J8	138	4498	1806
J9	50	423	1656
J10	84	1691	26

The remaining periods were not affected by flow breakdown.

An illustration of the method is given below for a particular day (6th February 2003 – 08:40 to 08:45 (Loop 5946A)) at junction 8. We begin with the speed plots at successive loops as shown in Figure 8.1 and Table 8.5: the loop nos. can be located on the plan in Figure 2.2, but they are encountered in ascending order by northbound vehicles.

Figure 8.1 Junction 8 Loop Speed Plots – 6th February 2003
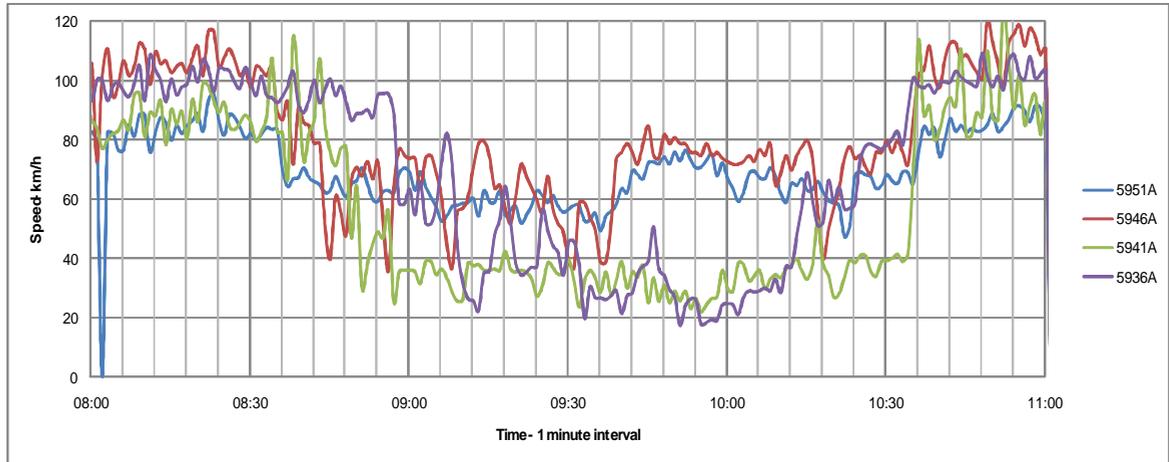
The rapid reduction in speed to 40 km/h can be seen at Loop 5946A (the red line) at approximately 08:45.

Table 8.5 Junction 8 Loop Speeds – 6th February 2003 – Breakdown Start

Period commencing	Average Speed (kph)				
	Loop 5946A	Loop 5941A	Loop 5936A	Loop 5931A	Loop 5920A
08:35	90	83	93	94	99
08:36	87	83	95	92	101
08:37	93	67	98	96	111
08:38	72	115	103	101	94
08:39	91	97	92	92	94
08:40	86	73	89	99	99
08:41	85	83	94	95	97
08:42	79	87	100	90	99
08:43	79	108	93	96	103
08:44	52	86	97	98	104
08:45	40	76	101	97	100
08:46	61	72	95	102	104
08:47	56	77	98	98	95
08:48	48	78	93	91	94
08:49	67	47	87	100	107
08:50	71	65	89	103	106
08:51	68	30	89	99	106
08:52	73	38	90	103	101
08:53	67	45	88	100	101
08:54	73	49	95	94	100
08:55	52	47	96	98	98
08:56	36	56	96	97	102
08:57	65	25	89	98	103
08:58	77	36	59	105	96
08:59	75	36	58	95	97

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Page 74

Here an obvious drop in speed can be seen at 08:44 (indicated by the blue shading). A drop in speed of 27km/h is evident, with the average speed dropping to 52km/h; considerably below the 60km/h guideline mentioned above. The breakdown can be seen to reach back to the upstream Junction 8 merge (Loop 5941A) at 08:49. From here a standing wave develops which continues even once the downstream Loop 5946A returns to more free-flowing speeds at 08:57.

The breakdown shown here was deemed to have ceased in the 10:30 to 10:35 period and this is the last period for which QDF was calculated. The green line in Figure 8.1, representing Loop 5941A located adjacent to the merge, shows a sudden increase in speed at roughly 10:34. Certainly the subsequent 5-minute periods have speeds for this loop consistently between 80km/h and 120km/h. This is shown in detail in Table 8.6.

Table 8.6 Junction 8 Loop Speeds – 6th February 2003 – Breakdown End

Period commencing	Average Speed (kph) Loop 5941A
10:25	41
10:26	41
10:27	36
10:28	34
10:29	39
10:30	39
10:31	40
10:32	42
10:33	39
10:34	42
10:35	70
10:36	114
10:37	89
10:38	92
10:39	80

It can be seen the speed reaches 70km/h at 10:35, so the 10:35 to 10:40 interval can certainly not be considered still to be affected by breakdown.

Although, as noted earlier, there is an element of judgment here, it was normally possible to make a robust assessment of the beginning and end of breakdown, following these principles.

8.3   Analysis of Breakdown Function (BDF)

For this purpose, only 5-minute intervals falling in categories a) and b) were used. The flow figures were grouped into flow bands with a band size of 10 vehicles (per 5 minutes). For each such band, we noted the total number of 5-minute intervals in categories a) and b), and of these the proportion where breakdown occurred (category b)). From this the probability of breakdown occurring at each flow level could be calculated. The flow value given to each band was that of the average of all the observations in that particular band. In some cases where a band did not produce any instances of breakdown, two or more adjacent bands were combined to create a larger group of observations which did include at least one occurrence of flow breakdown.

Initially, exponential trend functions of flow breakdown probability in relation to 5 minute flow were fitted in Excel, taking no account of the frequency of observations in each flow band.

Subsequently, a probit function was fitted using the MATLAB software. Essentially this means treating the probability of breakdown as a cumulative normal distribution $\Phi(z)$ and the task is to find the appropriate conversion $z = \dfrac{X - \mu}{\sigma}$ where X is the observed flow.

Assuming a binomial model of breakdown (using the total number of observations in any band, and the number of times breakdown is observed), MATLAB estimates the maximum likelihood parameters for the model $z = \alpha + \beta.X$, corresponding with $\beta = 1/\sigma$ and $\alpha = -\mu/\sigma$. The resulting parameters and t-statistics are given for each junction in Table 8.7.

Table 8.7 Probit Results

	α (= $-\mu/\sigma$)	β (= $1/\sigma$)	hence μ	σ
Junction 8	−10.7310 (−17.5)	0.0188 (14.9)	570.80	53.19
Junction 9	−7.3849 (−9.7)	0.0108 (6.5)	683.79	92.59
Junction 10	−10.0900 (−15.3)	0.0172 (12.5)	586.63	58.14

The fit for J9 was less good than for the other two junctions. This was not unexpected, given the limited number of occasions breakdown occurred at Junction 9 which was not due to downstream queueing.

As an illustration, the resulting probability plots and functions are shown in Figure 8.2 for Junction 10 (the bold entries in the table indicate where two or more flow bands have been combined). For example, there are 62 5-minute intervals where the 5-minute vehicle flow is in the range 520-530, and the average flow for these 62 cases is 523. Breakdown occurs in 11 out of the 62 cases, thus a probability of 0.177. Note that observations with flow below 380 have been omitted, since breakdown was never encountered below this level. The red line shows the probit curve fit, while the thin black line is the exponential curve.

Figure 8.2 Junction 10 BDF

Band	J10 Total	Average	BD	Probability	probit
380-390	837	384	0	0.0000	0.0004
390-400	866	394	0	0.0000	0.0008
400-420	**1459**	**409**	**2**	**0.0014**	0.0018
420-440	**1187**	**429**	**2**	**0.0017**	0.0048
440-450	430	444	5	0.0116	0.0100
450-460	395	455	2	0.0051	0.0156
460-470	308	465	11	0.0357	0.0234
470-480	267	474	5	0.0187	0.0332
480-490	219	484	12	0.0548	0.0481
490-500	154	494	7	0.0455	0.0672
500-510	132	505	9	0.0682	0.0926
510-520	96	514	11	0.1146	0.1228
520-530	62	523	11	0.1774	0.1555
530-540	21	535	2	0.0952	0.2062
540-550	19	544	3	0.1579	0.2535
550-570	**7**	**561**	**2**	**0.2857**	0.3534

Chart: Probability of Breakdown vs veh/5min. $y = 9.2695\text{E-}10e^{3.5564\text{E-}02x}$, $R^2 = 8.9500\text{E-}01$

While there is clearly an upward trend with increasing flow, what is noteworthy is that there is a non-zero probability of breakdown as low as 400 vehicles (equivalent to 1600 veh/lane/hr), while even as high as 560 vehicles (equivalent to 2240 veh/lane/hr), the probability is only around 30%. This is strongly indicative of the stochastic nature of breakdown, and provides some explanation as to why S-Paramics was not able to reproduce the required journey time variation.

The resulting flow breakdown probit probabilities by junction are plotted in Figure 8.3. The functions for Junctions 8 and 10 are reasonably similar. For Junction 8 the implied 50% probability is reached at 571 vehicles while for Junction 10 it is reached at 587 vehicles (ie approximately equal to 2330 vehicle/lane.hour). This similarity is slightly surprising, as the merge layouts are different – with Junction 8 having two separate merges and Junction 10 only having one. In addition, flow breakdown is usually caused at Junction 10 by a shockwave from downstream whereas this does not appear to be the case for Junction 8.

The function for Junction 9 is different to that for the other junctions. However, as the highest observed flow breakdown probability is only 3.4% and the fit is the poorest any extrapolation must be treated with caution.

Figure 8.3 Comparison of Probit Functions by Junction
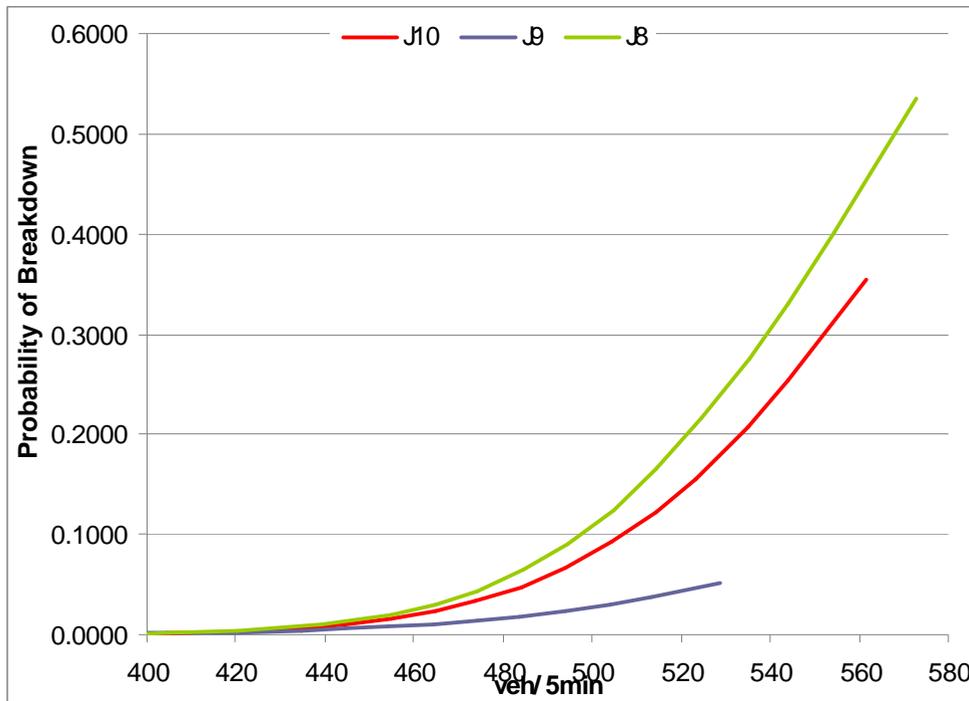
8.4   Analysis of Queue Discharge Flow (QDF)

It has been observed that during breakdown conditions there is an apparent loss of capacity, compared with the maximum flow that can be achieved in non-breakdown conditions. Moreover, this again appears to be a stochastic variable, and potentially location-specific. In some cases, QDF may be further affected by blocking-back from downstream junctions.

Hence, as with the BDF analysis, the QDF calculations were made separately for each junction, and it was checked to see if there was a significant difference in QDF between junctions. This analysis was carried out using all the 5-minute intervals which fell into category c), ie for all the periods from all the days analysed that were identified as being included in periods of flow breakdown.

The QDF values by junction for periods where there is no blocking back are shown in Table 8.8. These values are averaged over all 5-minute periods and the standard deviations and coefficients of variation (CV) relate to individual 5-minute periods.

Table 8.8: QDF Values

Junction	QDF Average	QDF SD	CV	Flow/hour/lane
10	447.1	34.4	7.69%	1789
9	445.4	38.7	8.69%	1782
8	442.1	35.5	8.02%	1769
Total	443.6	35.5	8.00%	1775

These values show that all three junctions operate with a QDF close to the average for all three junctions. The standard deviations and CVs are also very similar.

Junction 8 has an observed QDF closest to the mean for all three junctions. This is expected as two thirds of the total observations are from Junction 8.

Comparison of the mean QDFs for each junction with the mean for all the junctions using a Normal distribution test shows that in spite of the apparently small differences, for both the Junction 8 and 10 the mean QDF values are significantly different from the overall mean: this is due to the large samples involved. This suggests that individual QDF values should be used for each junction when there is no blocking back.

The lane/hour flow suggests that even in breakdown conditions, flows of 1,775 vehicles per lane per hour can be sustained.

QDF values were also generated for when blocking back from downstream was apparent. This would result in lower capacities at the junctions affected by the queueing back. Blocking back at Junction 10 from the downstream Junction 10a occurred only twice throughout the 81 days processed, so was not included due to this very limited sample size.

Table 8.9 shows the results for when blocking back from downstream influenced the QDF.

Table 8.9: QDF with Queueing Back Values

Junction	QDF Average	QDF SD	CV	Flow/hour/lane
9	403.0	46.0	11.42%	1612
8	367.1	57.8	15.74%	1469
Total	384.3	55.5	14.43%	1537

These values show a clear reduction in capacity once blocking back reaches the junction. Whereas throughputs with no blocking back were over 440 per 5-minute period, with blocking back they drop to approximately 400 for Junction 9 and to under 370 for Junction 8.

The difference between the observations at the two junctions was investigated further. It was considered that the following condition may be true:

$$QDF(n-1) = QDF(n) - merging\ flow(n) + leaving\ flow(n)$$

where n is the junction number increasing in the direction of flow

This assumes that the reduction of capacity at the upstream merge during times of blocking back will be directly related to the volume of vehicles leaving/joining the motorway at junctions between this junction and the junction merge at the head of the queue.

To check this condition a total of twelve days were analysed where queueing back was observed from Junction 10 as far back as Junction 8. The results of this are shown in Tables 8.10 and 8.11. Table 8.10 gives the observed average flows during the breakdown period. These can be compared with the values in Table 8.11 where a given QDF is adjusted to take account of the net traffic joining the queue downstream. For example, the QDF values in the second column of Table 8.11 are the derived values for J10m based on the observed J8m QDF value plus the net traffic joining at J9 and J10. In the final three columns of the table the ratio of the derived QDFs to the observed QDF at each Junction is given.

Table 8.10 Observed average QDF and Slip Road Flows during the period of Breakdown

Date	J8 QDF	J9 Offslip	J9 Onslip	J9 QDF	J10 Offslip	J10 Onslip	J10 QDF
11/02/2003	396	30	58	413	70	119	460
14/02/2003	345	16	61	385	39	98	444
14/03/2003	364	18	59	398	51	101	459
04/04/2003	357	16	61	401	49	112	470
24/04/2003	367	23	64	409	62	110	459
07/05/2003	376	24	66	421	71	126	484
08/05/2003	387	26	59	418	69	120	463
16/07/2003	385	25	52	411	75	123	464
18/07/2003	376	18	53	415	49	94	457
14/08/2003	360	19	43	408	50	110	467
04/09/2003	395	22	35	419	59	107	466
12/09/2003	356	17	65	401	43	102	465
Average	372	21	56	408	57	110	463
CV	4.39%	20.99%	16.50%	2.60%	21.08%	9.40%	2.02%
Max	396	30	66	421	75	126	484
Min	345	16	35	385	39	94	444

Table 8.11 Comparison of Observed and Derived QDF

	Derived QDF Based on			Ratio of Derived to Observed QDF		
	J8 QDF		J9 QDF			
Date	J9	J10	J10	J9	J10	J10
11/02/2003	424	473	461	1.03	1.03	1.00
14/02/2003	391	449	444	1.02	1.01	1.00
14/03/2003	405	454	447	1.02	0.99	0.97
04/04/2003	401	464	463	1.00	0.99	0.99
24/04/2003	408	457	458	1.00	0.99	1.00
07/05/2003	418	472	476	0.99	0.98	0.98
08/05/2003	421	471	469	1.01	1.02	1.01
16/07/2003	411	459	459	1.00	0.99	0.99
18/07/2003	410	455	459	0.99	1.00	1.01
14/08/2003	385	445	468	0.94	0.95	1.00
04/09/2003	408	456	467	0.97	0.98	1.00
12/09/2003	403	462	460	1.01	0.99	0.99
Average	407	460	461	1.00	0.99	1.00
CV	2.80%	1.96%	1.95%	2.24%	2.00%	1.11%
Max	424	473	476	1.03	1.03	1.01
Min	385	445	444	0.94	0.95	0.97

The analysis of these days shows clearly that the above assumption is applicable. On average, the ratio of observed and derived QDFs using the formula is exactly 1. The variability between days is very small, with all the ratios for each junction falling between 0.94 and 1.03.

8.5 Conclusions for BDF and QDF

8.5.1 BDF

Flow breakdown probabilities at merges can be forecast using the probit functions derived from the observed probability data. The function parameters used in modelling are set out in Table 8.7.

8.5.2 QDF

Where there is no blocking back the individual QDF values in Table 8.8 can be used for each junction.

Where blocking back occurs from junction n to the upstream junction n-1 the QDF at the upstream junction is derived from the following formulation:

$$QDF(n\text{ -}1) = QDF(n) – merging\ flow(n) + leaving\ flow(n)$$

where n is the junction number increasing in the direction of flow

Where blocking back extends to further upstream junctions this formula is applied progressively to determine the QDF of the upstream junction. In a case such as the northbound M6 in the West Midlands there the queue can extend from Junction 10 upstream to Junction 4a this could imply a very low QDF at the merge furthest upstream.

8.6 Transferability of BDF and QDF

The results in this Section must be considered highly specific to the stretch of motorway analysed. Nevertheless, the form of analysis is in principle transferable to other locations. In this section, we briefly discuss the special characteristics of the three junctions examined.

All three M6 merges analysed have no lane gain and there is no lane drop at the diverges at these junctions. Also the main carriageway standard is D3 with hard shoulders (the hard shoulders are mainly of standard width but there are probably substandard hard shoulders in the elevated area around Junction 9). However, the layouts of the merges have some differences, with Junction 8 having two separate merge lanes and Junction 9 having a very short merge. These differences may contribute to the differences between the BDF functions between the junctions. Also the high proportion of OGV on the M6 and the proportions of merging traffic may affect the functions. It would therefore be appropriate to validate the BDF functions for other merges on D3 motorways with no lane gain or drop at junctions where the proportion of OGV is low and for a range of merging proportions and merge layouts.

The QDF without blocking back varies marginally between the three junctions. This is probably due to the differences in merge layouts noted above and/or differences in the proportions of merging traffic and/or traffic composition particularly the proportion of OGV. The junction sample is too small to reach conclusions on the effect of layout, merging proportions and traffic composition, therefore additional junctions should be analysed to assess the effects of these factors.

The BDF functions and QDF values derived in this study should not be applied to flow breakdown at other types of merge, e.g. lane gain or drop, or at diverges or locations where flow breakdown is caused by a significant reduction in capacity, e.g. a mid link lane drop or a steep gradient, or where the main carriageway standard is other than D3. For these flow breakdown locations it will be necessary to validate the functions and values derived in this

study and if necessary develop specific functions and values. For main carriageway standards other than D3 it will be necessary to adjust the BDF and QDF functions to take account of the different numbers of lanes. This adjustment could use a simple factoring based on the number of lanes but it would be necessary to validate this at appropriate flow breakdown locations.

8.7 Speed-flow for Non-Breakdown Periods

In the case of non-breakdown periods the Aggregate Model assumes a conventional mean speed-flow relationship with the addition of variability around this mean.

There are a number of recommended equations for the relationship between speed and flow on motorways (COBA for instance), but these do not provide any evidence of variability. We have therefore estimated both mean and variability from MIDAS loop readings on the M6 for three links. These average relationships are based on a set of observations at some location. For the J10 reading they are loop detector readings at 5979A (a small proportion of data come from 5984A when 5979A failed to work). For J8d – J8m the data is derived from 5920A, and for J9 the source is 5955A.

Figure 8.4 provides the example for J10.

Figure 8.4: 5 Minute Speed Flow Plot for J10 distinguishing Breakdown and Non-Breakdown Conditions
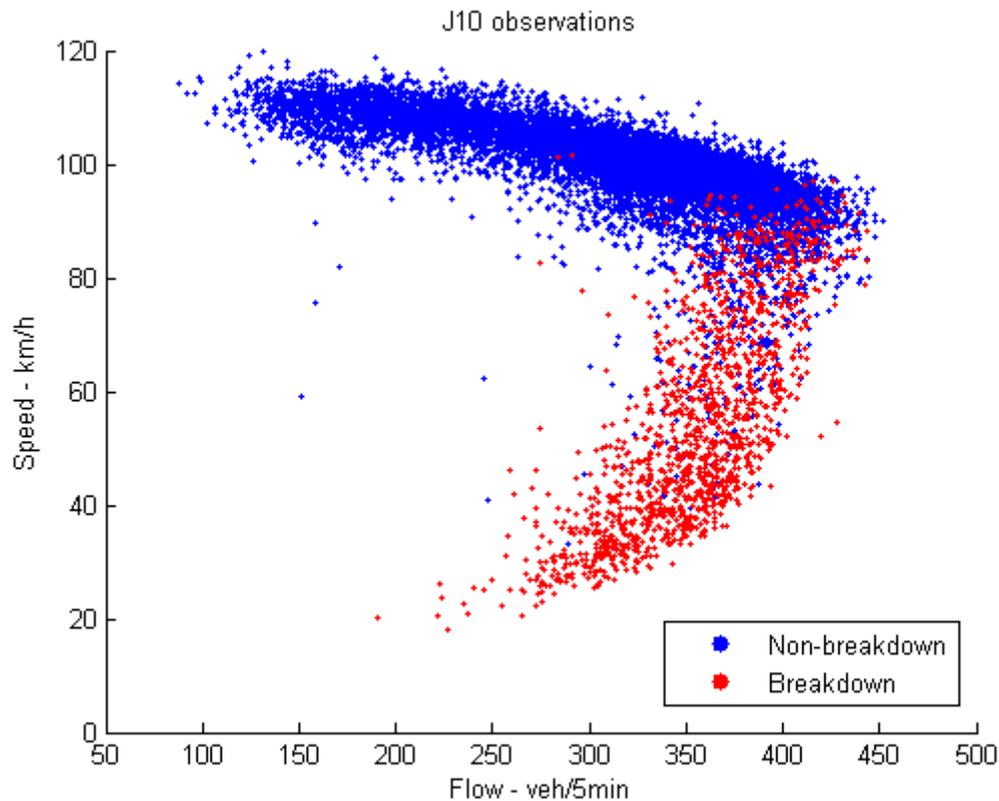
Using the non-breakdown data illustrated in the figure regression analysis provides the constant and slope of the speed-flow relationship. It also reveals the standard error of the equation, which is a measure of the variability in speed.

For the three available data sets the parameters in Table 8.12 are found, where flow is the number of vehicles per 5 minutes, and speed is in km/h.

Table 8.12 Speed/Flow Regression Analysis

	Constant	SE coeff	Slope	SE coeff	SE equation
J9	123.0	0.29	-0.0797	0.00081	4.94
J8	121.2	0.17	-0.0611	0.00075	3.34
J10	122.9	0.17	-0.0718	0.00053	3.91
J10 <300 veh	117.9	0.28	-0.0494	0.0012	3.61
J10 >300veh	127.6	0.67	-0.0851	-0.0019	3.91

This demonstrates a small difference in the slopes and constants for the 3 sources. With a sample of over 6000 5-minute periods these differences are significant at the 95% level. Examination of the data for J10 shows that the relationship is slightly curved with the slope increasing as vehicle flow increases.

However this simple interpretation is only part of a more complex story. In adopting a standard deviation (or CV_s, the Coefficient of Variation of speed) to use on a sequence of links we must also consider:

- whether CV_s varies with average speed?

  As we are interested in the variation a driver might expect travelling the same link on a number of days the CV can be expected to be low at low levels of flow, when an individual driver is unaffected by other vehicles in his choice of speed. As flow increases there is more disruption to his journey and CVs is expected to rise. Empirical data will also incorporate the differences between drivers and those due to vehicle mix. Even at low flows there will still be variation due to external factors such as the weather.

- whether CV_s varies with distance over which the speed estimates are derived?

  The CVs can be expected to fall with increasing distance.

- whether there is correlation between adjacent links?

  If there is no correlation then for a set of links comprising a journey then the variance of travel time for the journey is simply the sum of link variances

Using the 67 day sample we consider each 5min period during the day (0600-2200), using the classification in Section 8.2. For those 5 minute periods where at least 30 of the 67 observations are non-breakdown, we calculate the standard deviation of the speed for that period, and these are plotted against the average speeds in Figure 8.5. A distinction is made between pre 1500 periods, where the proportion of heavy vehicles is significantly higher, and post 1500 periods.

Figure 8.5 shows clearly the lower journey time (higher average speed) for the post 1500 periods, and also confirms the tendency for standard deviation to rise as speed falls. Since the distance is approximately 11.18 km, the range of 440 to 380 seconds corresponds with speeds of 91 to 106 km/h.

Figure 8.5: Standard Deviation of Travel Time in 5min periods, Camera 1a-4
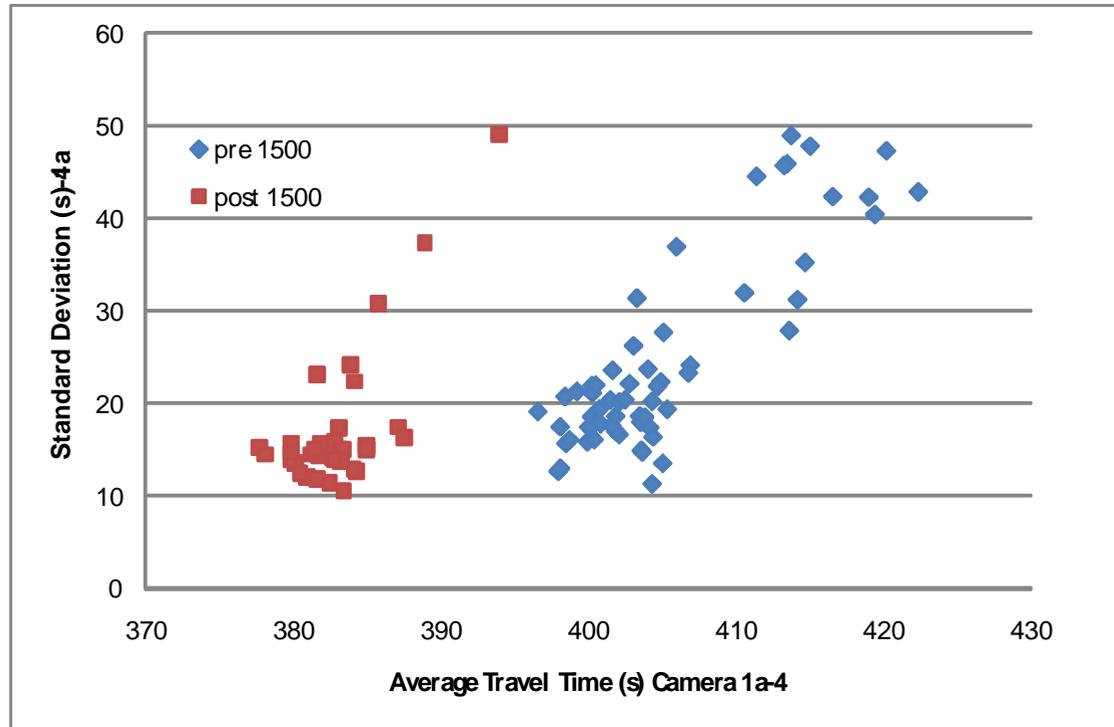
If, in line with the figures found in the results in the last column of Table 8.5, we assume a standard deviation for the speed-flow relationship of 4 km/h, then the modelled standard deviation of travel time (Camera 1a-4) can be calculated analytically (by summing travel time variances on each link). At a level of flow giving mean travel speed of 100 km/h this calculation gives a standard deviation for camera 1a-4 of about 8 secs. This can be compared with a minimum of 10 secs found in the 67 day sample as shown in Figure 8.5.

Judging from evidence in previous studies of travel time variability, a positive correlation between adjacent link travel times is probably a contributor to the higher standard deviation observed. For the configuration of links used in the Aggregate Model a standard deviation of 5.4 km/h for speed on a link would be required to generate a travel time standard deviation of 10 secs assuming no correlation (or 10.2 km/h to achieve 20 secs).

For the purpose of this study a simple relationship of the form shown in the graph can be used to calculate standard deviation for the journey from camera 1a-4. This implies a CV (of travel time) in the range 0.04-0.06 for a distance of 11.3 km. The result is not transferable to situations where the distance is different.

Note that of the total travel time variance found in the 192 5min periods the non breakdown periods contribute less than 1% total variance. Whilst further research and analysis to develop a more sophisticated model of variability on non breakdown periods could be undertaken it is clear that the main focus in understanding TTV should be on those periods experiencing breakdown to some degree.

9   AGGREGATE MODEL VALIDATION

9.1   Introduction

In this Section we describe the application of the Aggregate Model and assess its success in producing acceptable estimates of the profile of travel time and its variability. The model follows the specification in Section 6 and the data and parameters described in Sections 7 and 8.

For each entry and exit, the model requires input data relating to the profile of demand and its variability. As discussed in Section 7, the overall demand is divided into two components – total daily demand, and the profile during the day (defined as 0600-2200). Because variation by day of week was found for at least some of these input components, the Aggregate model is run separately for each of the five weekdays. The Demand data relates to 64 weekdays, and the validation data, based on ANPR records, is also based on these same days.

The data that is used for validation are the estimates of mean travel time between ANPR cameras 1a and 4 in 5 minute periods for each day of the week. – see Figure 9.1. Note that the general pattern of travel times by day is similar to that of Figure 3.2, but the sample of days used is different.

Figure 9.1   Average Travel Time between Cameras 1a and 4 by Day of Week (seconds)
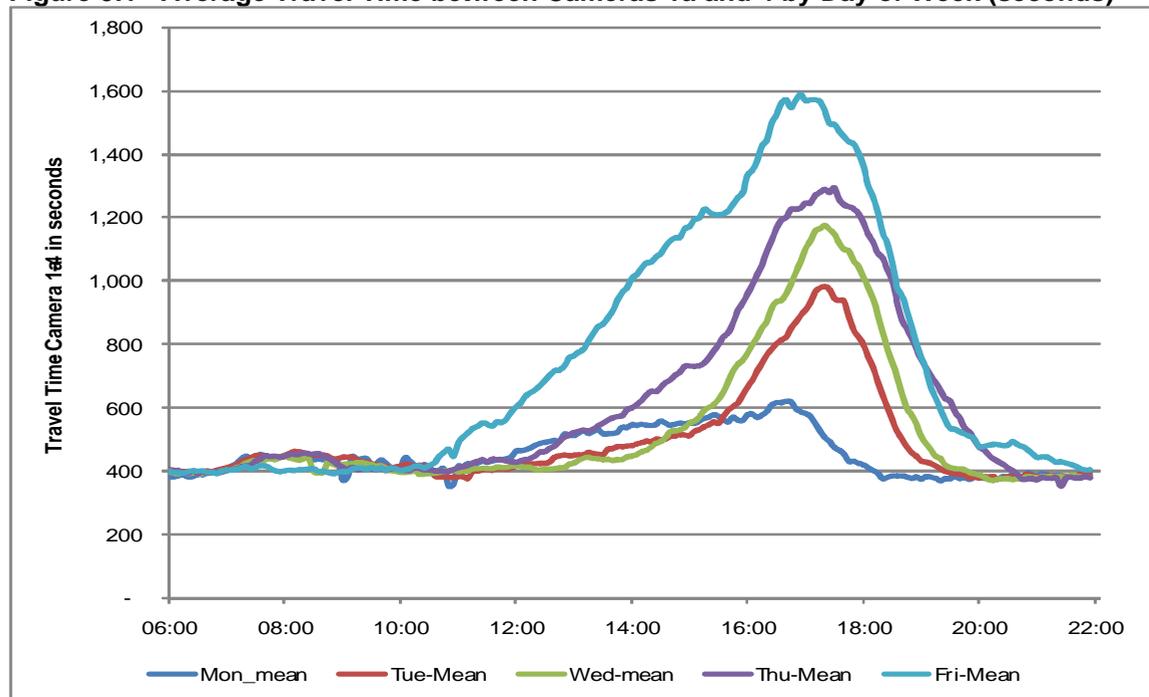
In assessing the model validation, it needs to be recognised that the observed travel time statistics are a sample. With the number of observations for any particular day of week between 7 and 16 the 95% confidence interval can be nearly ±20% for mean travel time and ±50% for its standard deviation. As an illustration, Figure 9.2 shows the 95% confidence intervals for Friday.

Figure 9.2   95% Confidence Interval - Average Travel Time (Friday)

Motorway Travel Time Variability—Final Report
Hyder Consulting (UK) Limited-2212959   **Page 84**
c:\documents and settings\cdga97640\my documents\motorway ttv\mttv final report 27 oct.docx
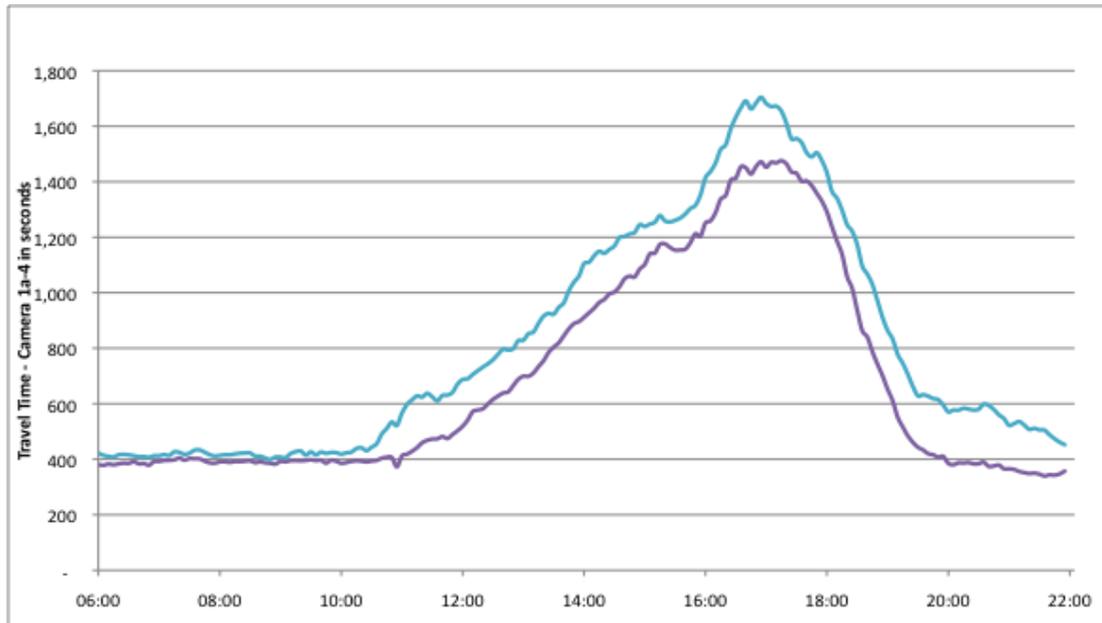
Figure 9.1 shows a small increase in mean travel time during the am peak, with the major increase in the pm peak. The maximum average travel time is lowest on Monday rising throughout the week and reaching a peak on Friday. While the highest time here is about 1,500 seconds (i.e. 25 minutes), Figure 9.2 shows that the upper bound of the 95% confidence interval is around 1,700 seconds (28 minutes). Since even at 80 km/h (50 mph) the time taken to cover the section 1a-4 is only 8.5 minutes (= 510 seconds), it is clear that the significantly higher times in these diagrams are primarily associated with flow breakdown.

The corresponding variability pattern is shown in Figure 9.3: the pattern is similar to that of Figure 3.4, but the sample of days is reduced. Day to day variability of Travel Time between cameras 1a and 4 increases from about midday reaching a peak around 1700-1800. Sometime after 1800 the standard deviation falls rapidly. Monday returns earliest to the low variability found in non-breakdown periods, with other days following in order.

Figure 9.3   Day to Day Variability by Day of Week -Standard Deviation (seconds)
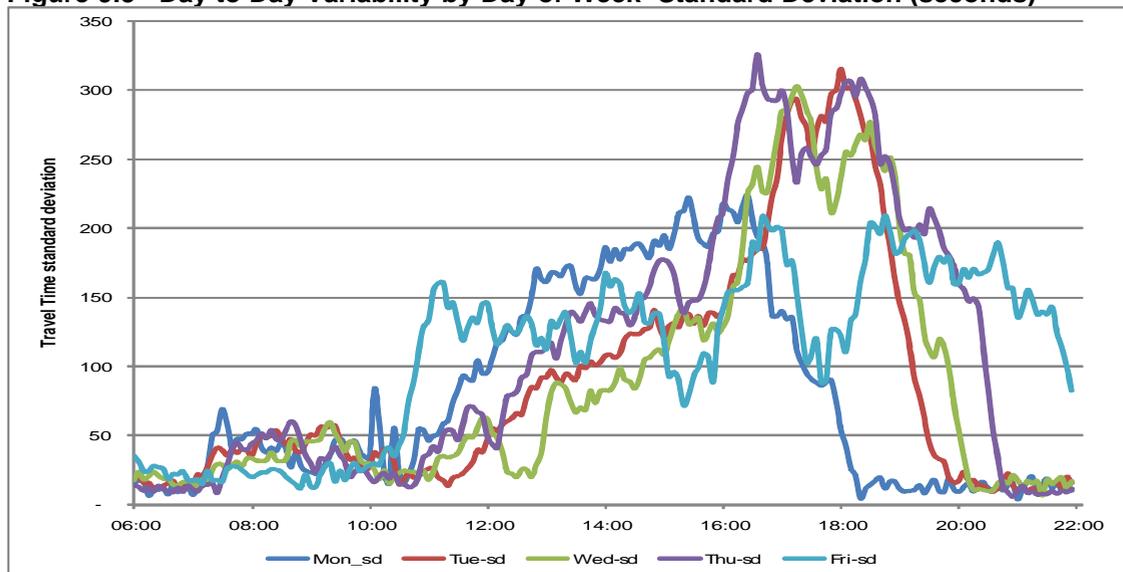
Again, as an illustration, Figure 9.4 shows the 95% confidence intervals for Friday.

Figure 9.4   95% Confidence Intervals - Standard Deviation of Travel Time (Friday)
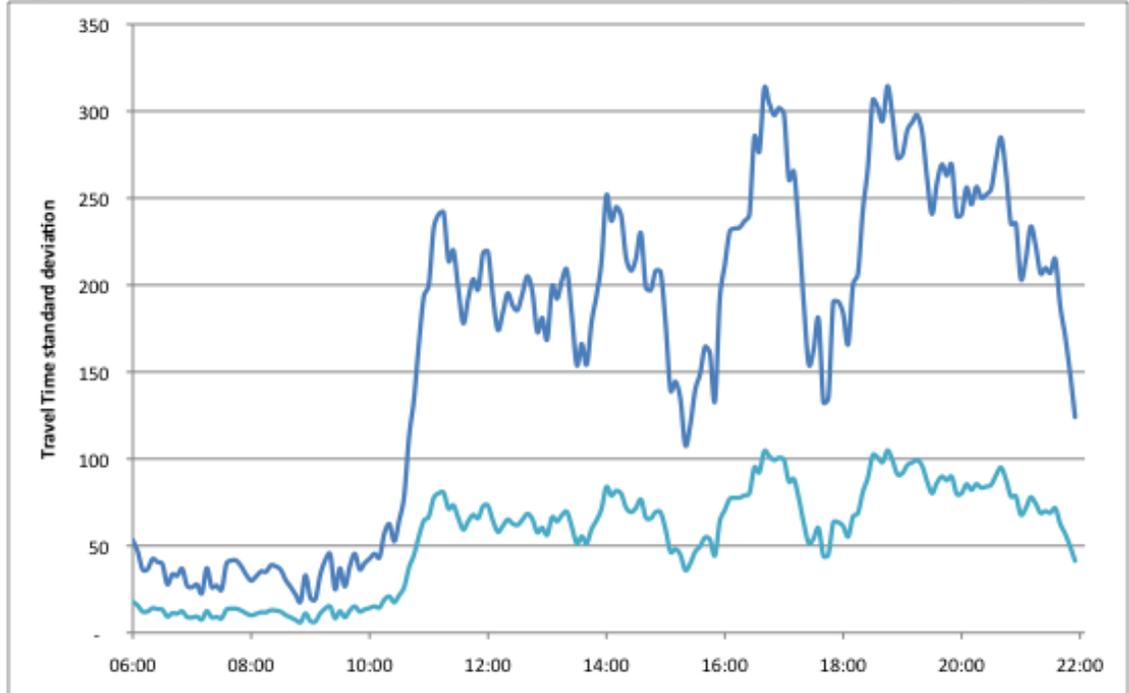
Our preferred indicator of TTV is the coefficient of variation (CV). Based on Figures 9.1 and 9.3 we present the observed CV data by day of week in Figure 9.5:

Figure 9.5   Day to Day Variability by Day of Week - Coefficient of Variation
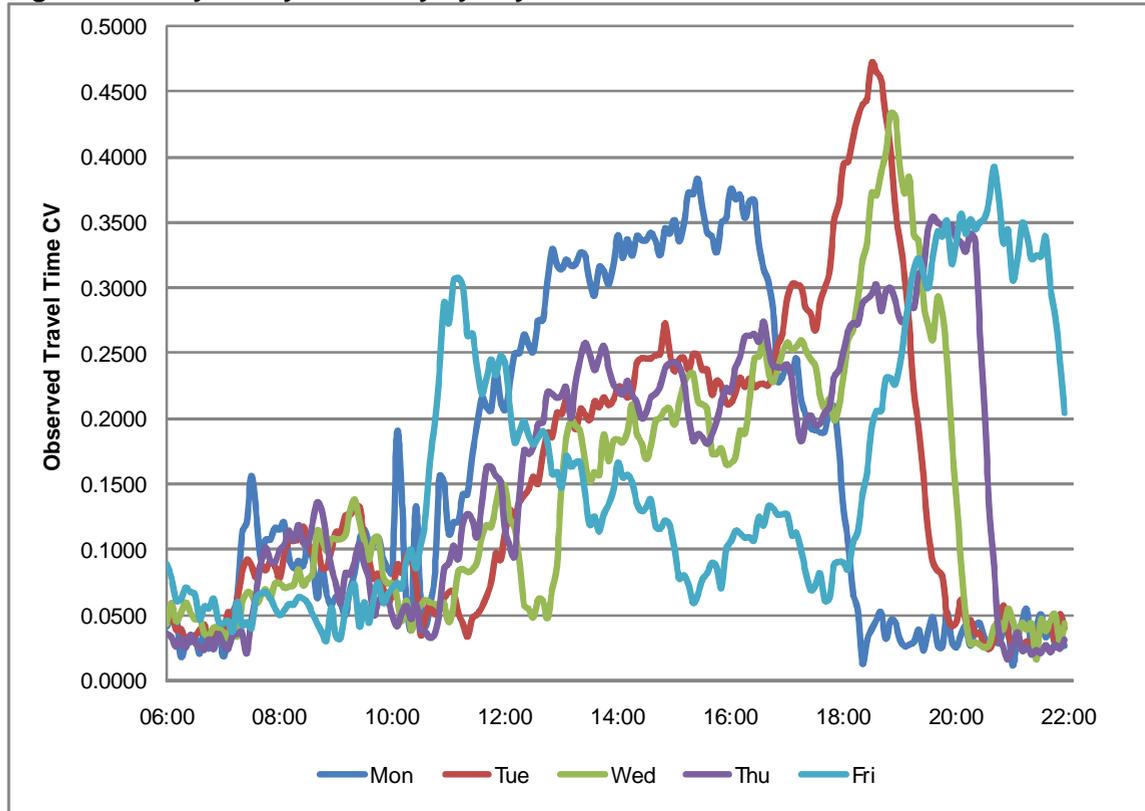
Tuesday, Wednesday and Thursday follow a similar pattern peaking above 0.35. Monday and Friday are noticeably different and are the subject of further commentary later.

The Aggregate model is generally run for 100 simulated days, and the Figures in this Section are presented on that basis. Tests using 1000 days removed a certain amount of the lumpiness in the graphs but did not materially alter the results.

9.2 Understanding the Aggregate Model

9.2.1 Background

It will be evident from the discussion in Section 4 that we have concluded that the main source of TTV appears to be standing wave flow breakdown. The Aggregate model treats flow breakdown as a stochastic phenomenon which is more likely to occur as demand (relative to capacity) increases, as was shown in Figure 8.3.

9.2.2 Breakdown Probability

Although the probability of breakdown in any one 5-minute interval is generally low, the cumulative likelihood of it occurring at some time during sustained periods of (relatively) high demand can be high. We begin by illustrating how the process operates.

For breakdown, the relevant demand is the sum of the two link demands at the merge. For example the estimated mean profile for the J8 merge demand is shown in Figure 9.6 (the Tuesday and Wednesday curves generally overlap, based on the demand analysis in Section 7). The merge demand for a 5 minute interval is calculated by summing the M6 demand and M5 demand. Section 7.5 describes the derivation of demand for periods in which only flow (as opposed to demand) is observed. The method leads to the straight line segments on the demand profile for Friday.

Figure 9.6: Demand profile for J8 merge by day of week
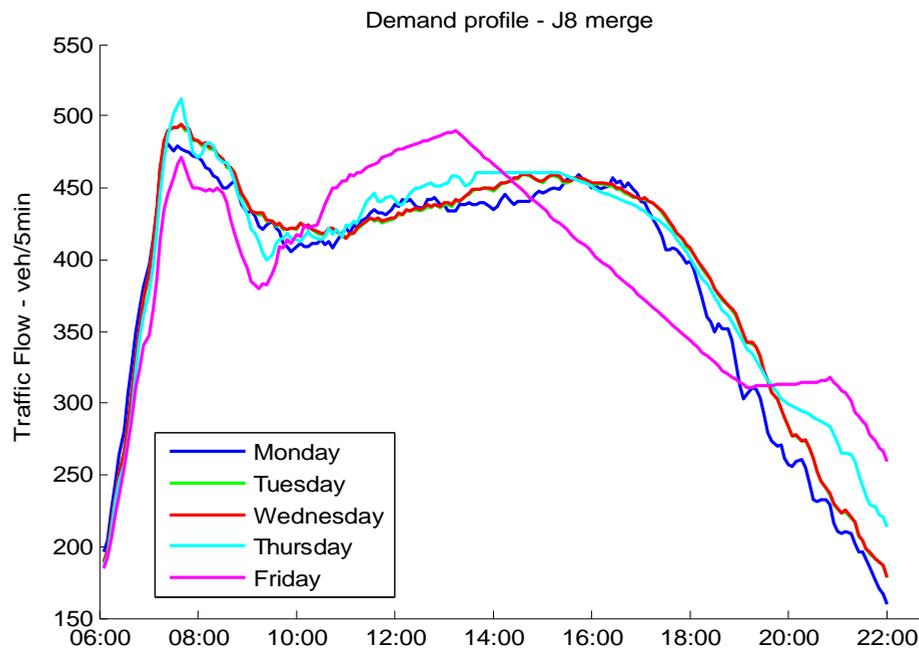
The general pattern is of two separate peak periods, before and after 10.00. In respect of illustration we concentrate initially on the post 10.00 period, and we take the Thursday example.

If, in the first place, we ignore the 5 minute demand variability, then we can use the BD function to predict the probability of breakdown in each 5-minute period for a particular merge. We can also show the cumulative probability (P_t) that breakdown has occurred by time t. This is calculated in the following way:

At the beginning of the relevant time intervals set $P_0 = 0$. For a given demand flow we can calculate the probability of breakdown in any period t which we write as π_t. Assuming that once initiated breakdown survives, then the cumulative probability P_t that breakdown has occurred by time t is obtained from the two equations:

$$\rho_t = \pi_t.(1-P_{t-1}),$$

$$P_t = P_{t-1} + \rho_t$$

Note that ρ_t is the probability that breakdown first occurs in period t (taking account of the possibility it may have occurred in a previous period).

Applying this approach to the mean demand profile for junction 8 merge we obtain Figure 9.7.

Figure 9.7   Probability of Breakdown using average Thursday merge demand at J8
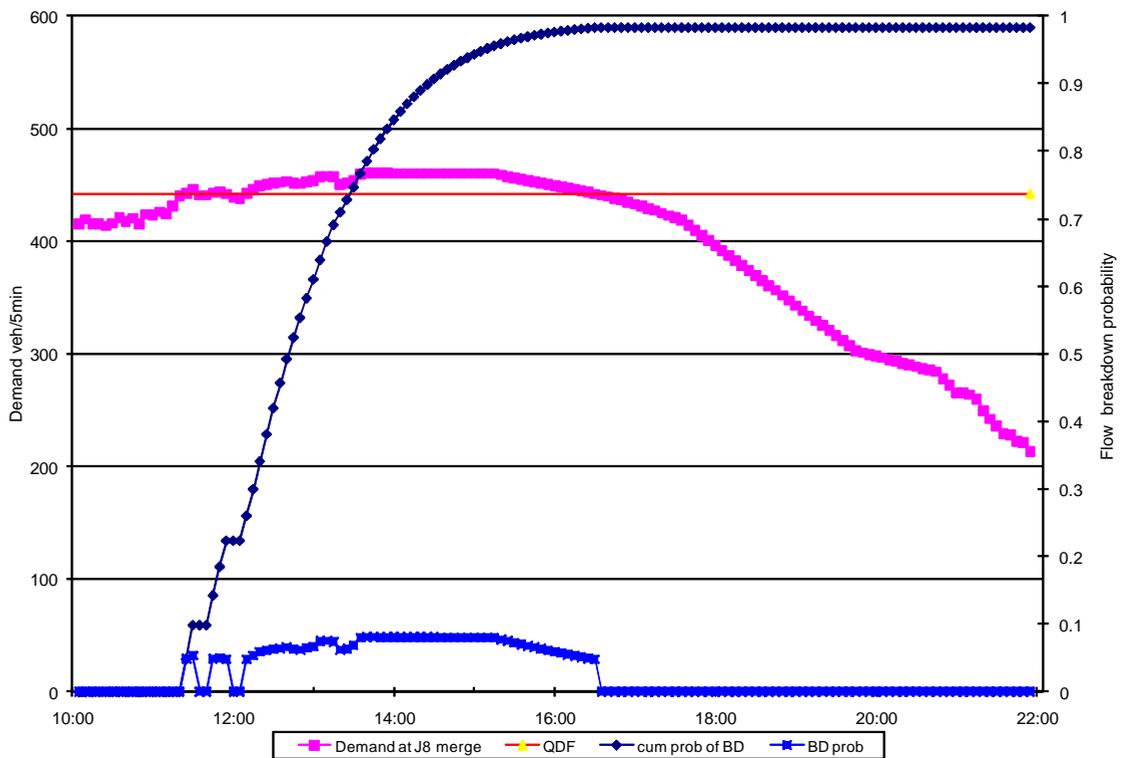
This shows a) how the breakdown probability varies with the merge demand and b) how this transforms into the probability of breakdown actually occurring sometime during the day. It can be seen that the probability of breakdown occurring in any one 5-minute period is never more than 10% but, depending on the length of time that demand remains higher than about 420 vehicles per 5-minute period, the overall probability of breakdown can become very high.

In practice, the Aggregate model generates the level of breakdown by simulation: a) by repeating the process for a number of days and b) sampling the demand randomly in each 5-minute interval.

Given the (Probit) form of the breakdown function, it is also possible to derive an analytical solution for π_t incorporating the demand variation, along the following lines:

Breakdown is initiated by a comparison of merge demand with stochastic capacity (BDF). The two components of merge demand, as well as capacity, are assumed to be Normally distributed, i.e.:

$$d_{1,t} \sim N(D_{1,t}, \sigma_{1,t}{}^2); \; d_{2,t} \sim N(D_{2,t}, \sigma_{2,t}{}^2); \; c \sim N(C, \sigma_c{}^2)$$

Then the Probit form for breakdown becomes the probability that $d_{1,t} + d_{2,t} - c > 0$, and assuming independence between variables, this is given by

$$\pi_t \sim \Phi(D_{1,t} + D_{2,t} - C, \; \sigma_{1,t}{}^2 + \sigma_{2,t}{}^2 + \sigma_c{}^2)$$

where Φ is the cumulative normal distribution.

The simple probability of breakdown at a particular time t - π_t - can be transformed, as before, into P_t, the cumulative probability that breakdown has occurred by time t, and the two alternative approaches (analytical and simulation) can be compared.

We can go on to see how well these alternatives compare with the actual breakdown experienced on a typical day. For these, the observed data also need to be transformed into cumulative probabilities (accepting that the sample for any one day is limited). Note that there are marginal problems in doing this as on some days breakdown has occurred twice with an intervening period of recovery.

Such a comparison will provide an initial idea of how well the BD function works. However, even when breakdown occurs, the consequences depend on how early in the period of (relatively) high demand it occurs: the later it occurs, the shorter the queues that it generates, and hence the lower the impact on travel time. This means that our judgment of how well the cumulative breakdown curve fits needs to reflect the importance of getting the distribution of the onset of BD correct. Hence we now need to discuss the factors affecting the duration of breakdown.

9.2.3   Breakdown Duration

Once breakdown is initiated, it will continue until the implied speed is no longer below the corresponding non-breakdown speed (see Section 6). Essentially the "speed" (or time to clear the link) is governed by the length of the queue. The queue will continue to grow as long as demand remains above QDF: from the time demand falls below QDF, the queue will diminish, but the remaining time under breakdown conditions will depend on the maximum length of the queue. Hence, the timing of the onset of breakdown in relation to the demand profile is critical.

Returning to the previous example based on the Thursday mean demand profile for J8, Figure 9.8 shows how the queue is affected by the time at which BD occurs.

Two alternative times are shown, one (at 12.10) early in the period of high demand, and the other one (at 15.00) after the peak of high demand. It can be seen that the impact on the queue for these two alternatives is quite dramatic, in terms of the queue length and (hence) the time to return to normal conditions. It needs to be stressed that BD could occur at any time in the time range of higher demand, which in this example is 1115 to 1630. In both cases the maximum length of the queue occurs at the same time, which is when the merge demand falls below QDF. Note that the curve here does not make allowance for the stochastic nature either of demand or QDF.

Figure 9.8   Duration of Breakdown using average Thursday merge demand at J8
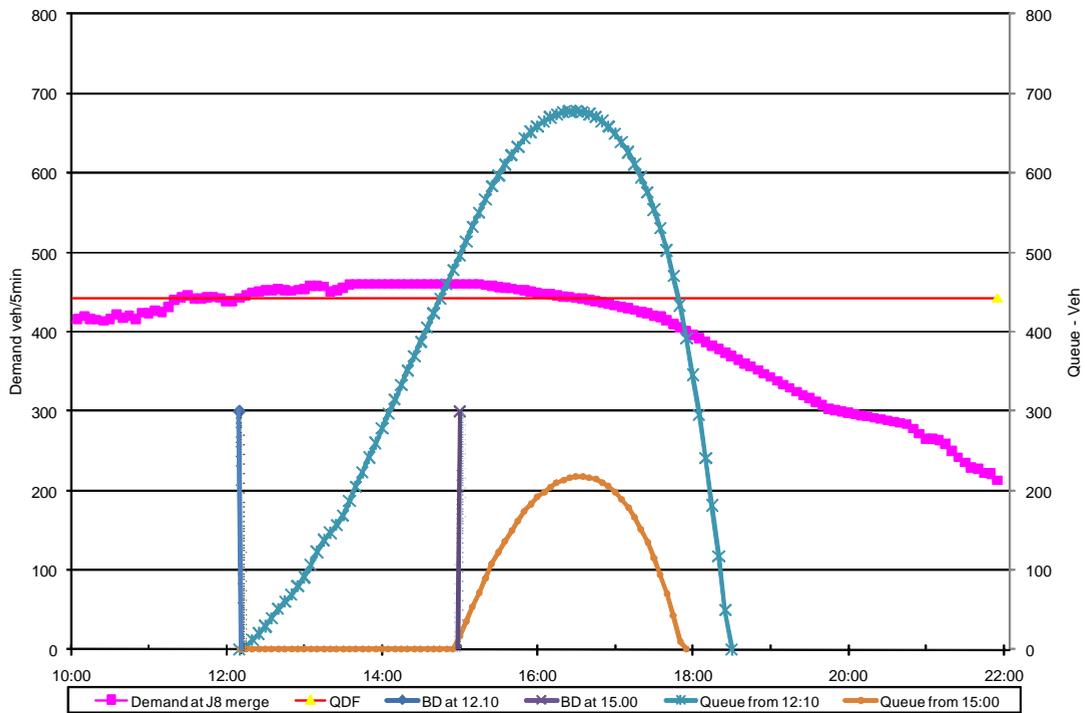
When we turn to the pattern that might be expected for a **set** of days, we will observe something like the 7 lines shown in Figure 9.9, where the times of breakdown are arbitrarily assumed to be evenly spaced at 30 minute intervals. In addition the mean queue length ("Average Queue") and its standard deviation ("SD Queue") are also shown. If we define delay as the extra travel time experienced by vehicles arriving in period t then delay is proportional to the length of queue, so that travel time also follows the profile of queue length.

Figure 9.9: Dependence of Queue Length on Time of Onset of Flow Breakdown
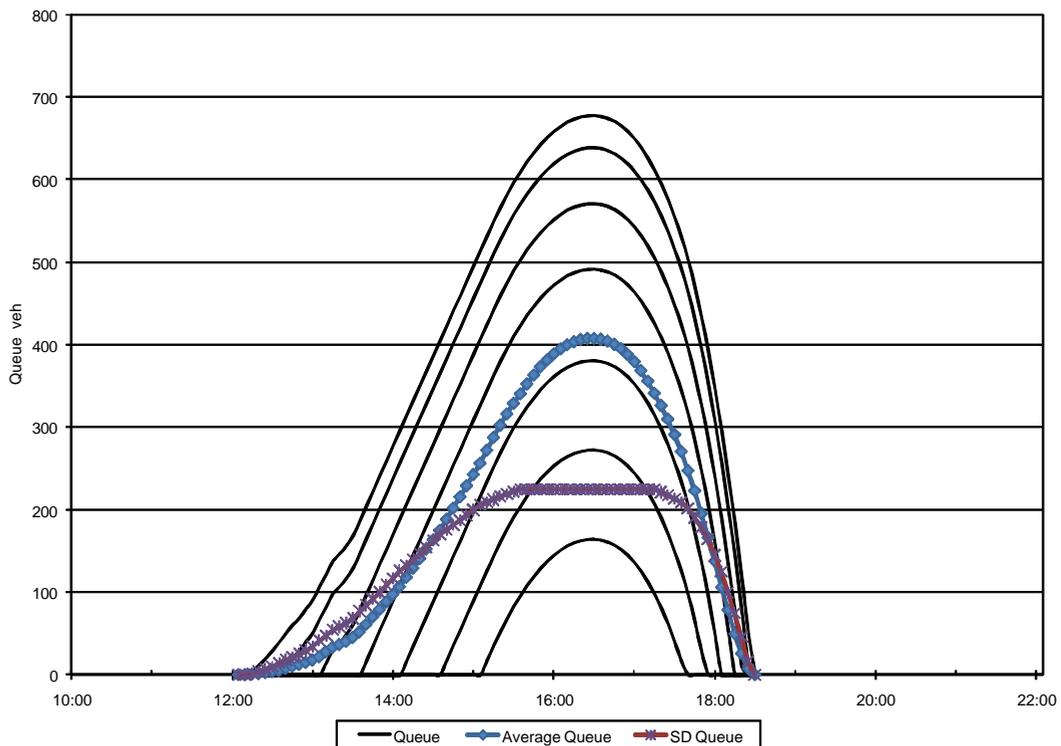
If mean QDF is higher then the maximum height of each curve will be lower and the duration will be shorter. The QDF that is relevant in the above case is main line QDF, which is dependent on total QDF (reported above) and the main line share.

In the graph the profile of mean queue length (implicitly, mean delay) is shown by a curve using diamond markers. Note how in the initial phase the curve rises slowly. In the final phase the decline is faster. This is explained by the fact that demand is falling faster in the evening than it is rising during the onset of breakdown. This difference also explains the asymmetry of the rise and fall of the observed mean travel time curves (see Figure 9.1)

Note also that, although the breakdown periods are evenly spaced, the mean profile does not have the same shape as the median day profile. In the early phase of breakdown the mean profile rises slowly reflecting the increasing number of breakdown days. During the middle of the breakdown period the mean travel time shape is similar to the shape of individual days.

Another important interpretation of this graph is concerned with the profile of standard deviation over the breakdown period (shown as the lower asterisk marked line with a flat top). The closer the lines in this diagram, the lower will be the standard deviation of travel time. The closeness of the lines is determined by the proximity of breakdown times. The latter is strongly dependent on three factors

- The breakdown variance referred to in the last section ($\sigma_1 + \sigma_2 + \sigma_c$ - variance of the main line, ramp and BDF respectively)

- The day to day variance of total demand

- The rate of growth of demand during the potential breakdown period.

As the variances fall then the onsets of breakdown become closer. In the extreme case if both variances are zero then breakdown occurs at the same time (same level of demand) each day. If demand grows very rapidly (as it does on Friday at some times) then the breakdown times will also tend to be condensed.
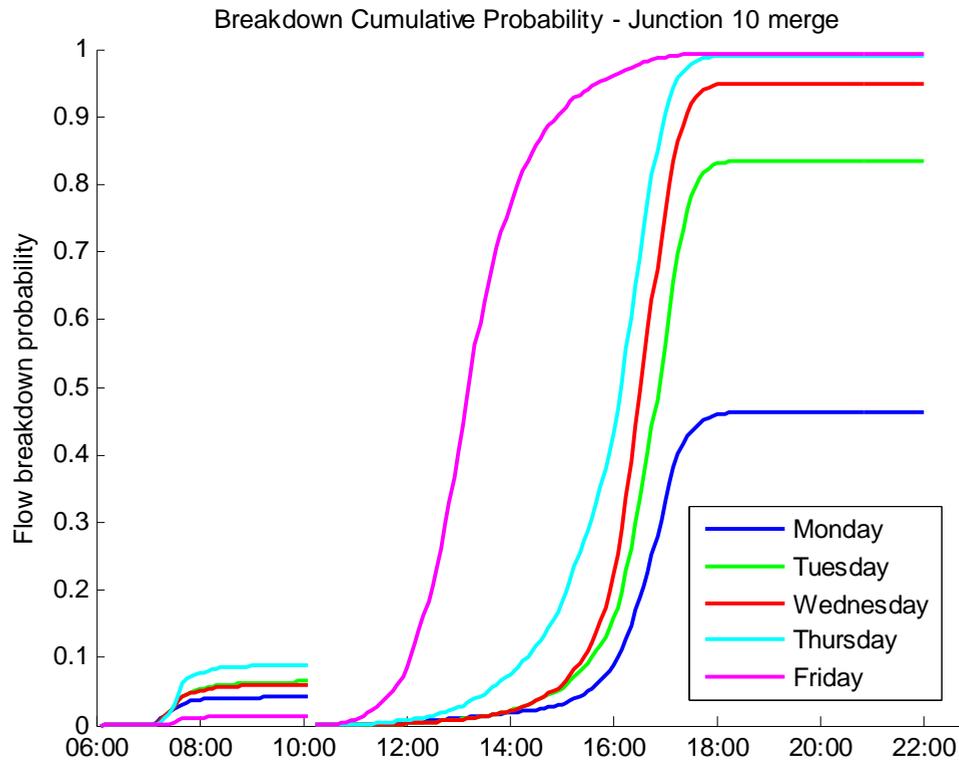
The graph is based on the Aggregate Model using the average demand profile and QDF without random effects. A graph including the variability of QDF and the 5min variability will be essentially the same but with spikes recognising the individual 5 minute variability of demand minus QDF. These spikes are seen in the observed travel times. In addition the observed lines for a set of days are not always well ordered (in the sense that day x always remains above day y and below day z). This is because a day's demand profile is not always a simple factor of the average demand profile.

9.2.4 Propensity for Breakdown

Using the analytical cumulative probability of breakdown formula of section 9.2.2 and the graphs of average merge demand by day of week it is possible to calculate the overall probability of breakdown occurring at each junction by day of week, as shown in Figures 9.10. The calculations are undertaken separately for the am and pm peaks, though in fact there is a small probability that breakdown does not end at or before the end of the am peak (1000 h).

Figures 9.10: Propensity for Flow Breakdown by Day of Week for Each Merge

Breakdown Cumulative Probability - Junction 8 merge

Breakdown Cumulative Probability - Junction 9 merge

Breakdown Cumulative Probability - Junction 10 merge
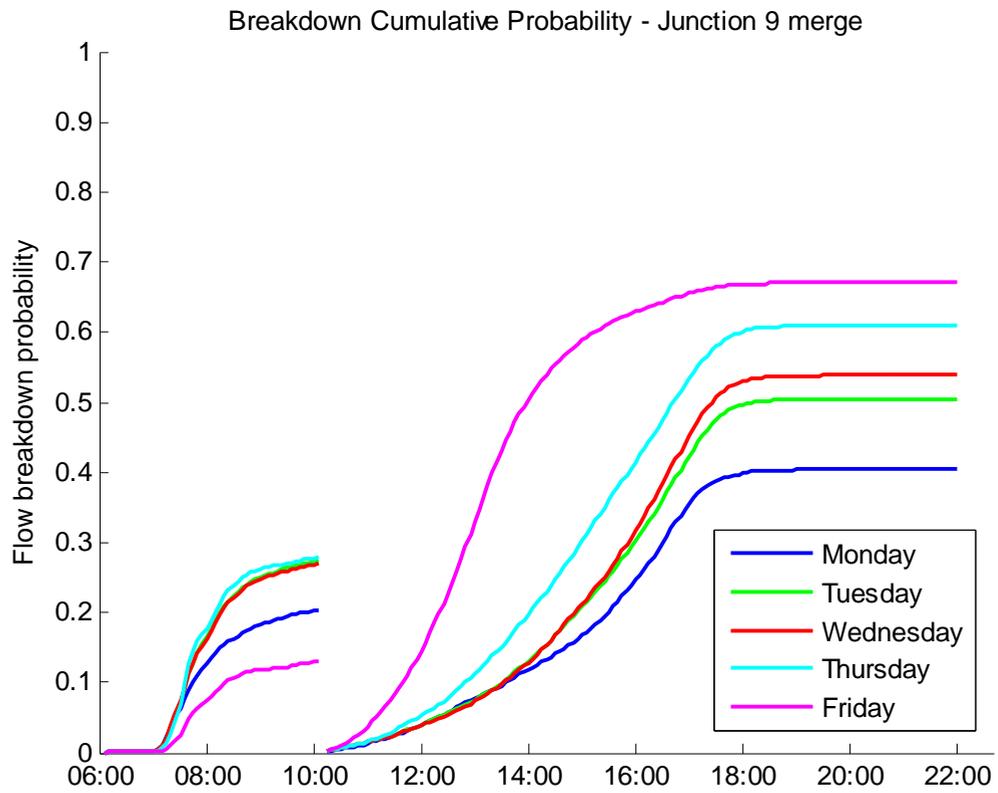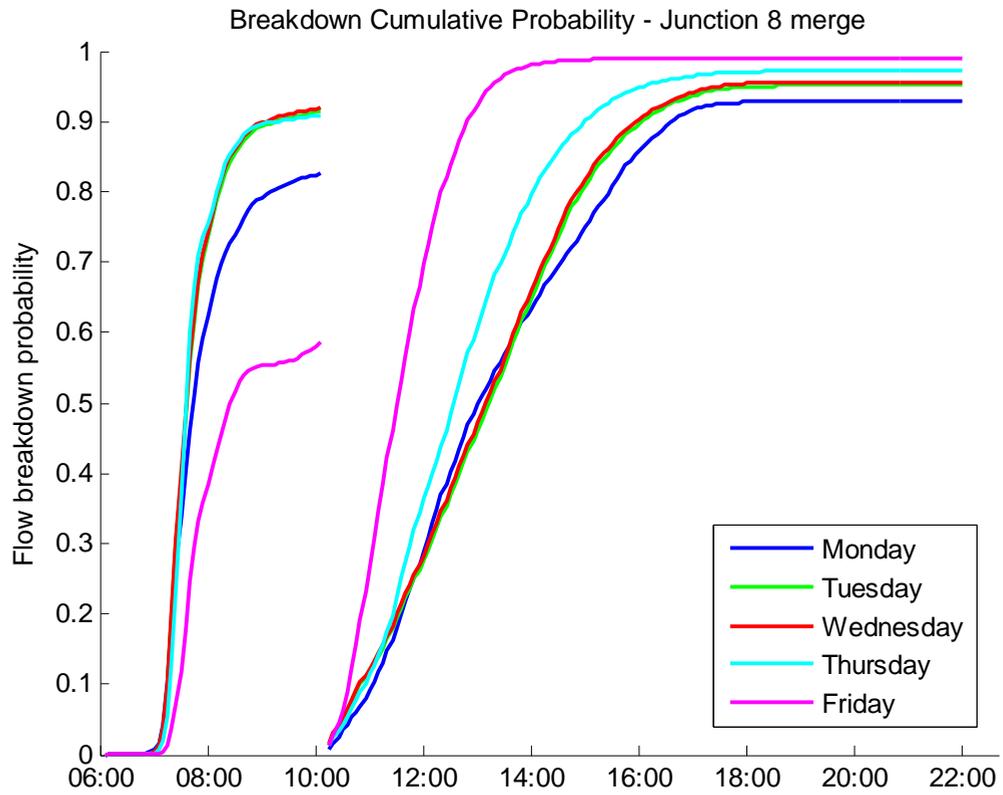
These cumulative breakdown probability graphs illustrate a number of features:

- The small probability of breakdown in the am peak for J10.

- The earlier breakdown at J8 compared to J10

- The much lower propensity for breakdown at J9 am and pm

- The earlier breakdown on Friday for J8, J9 and J10

In order to illustrate the comparison between the analytic prediction of the proportion of time a junction is in breakdown and the prediction from simulation Figure 9.11 shows the situation for the merge at J10. In the pm period the analytic curve (P_t - the cumulative probability that breakdown has occurred by time t) rises smoothly and reaches a plateau. In comparison the modelled shows both the rise and decline during the pm period. The analytic curve assumes that breakdown is maintained, whereas the modelled curve includes the ending of breakdown (which in some cases occurs soon after the initial breakdown). It is the rise that can be compared to the analytic P_t - the cumulative probability that breakdown has occurred by time t. The peak of the two curves is similar for all days. The rise in the proportion is similar for all days except Friday. On Fridays for this merge the reason for the later delay at J10 is because on this day of the week earlier flow breakdown at junction 8 reduces demand arriving at J10.

Figure 9.11: Propensity for Flow Breakdown by Day of Week (Junction 10)
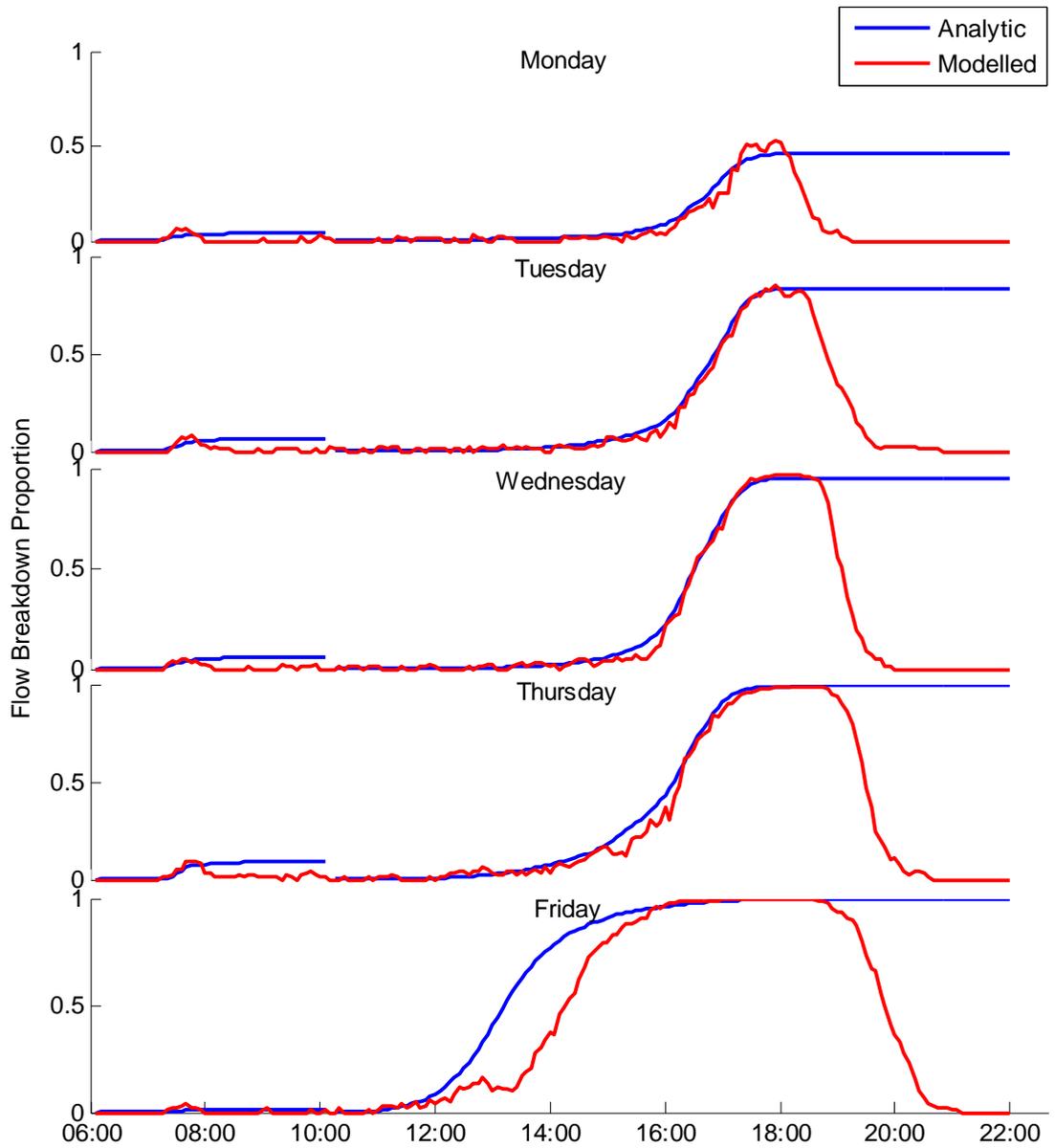
9.3 Model Results

9.3.1 Breakdown Proportions

The Breakdown Proportion for a 5min period t, denoted as R_t, is defined as the proportion of days that a given location is in a breakdown condition in period t. The Breakdown Proportion is a pivotal statistic in forecasting travel time and total delay caused by breakdown. The Aggregate Model calculates it in the following way:

For any simulated day d, we have the (stochastic) merge demand at time t, from which we can calculate the breakdown probability π_{td} using the probit function. In order to decide whether breakdown actually occurs, we need to sample from a binomial distribution with mean π_{td}. If we sample 0, then breakdown does not occur, we set the breakdown status variable B_{td} to 0, and we move on to consider the next 5min period. On the other hand, if we sample 1, then breakdown occurs on that day and we can set the breakdown status variable B_{td} to 1. Once breakdown occurs, the status for all subsequent 5min periods t is also set to 1 until the end of breakdown condition is met (see Section 6.2.2), when it is set back to 0.

The upshot is that for each 5min period in each simulated day there is a breakdown status vector with values of either 0 or 1. Separate status vectors are calculated for each merge location. If there are N simulated days, then R_t is calculated as:

$$R_t = 1/N \; \Sigma_d \; B_{td}$$

Note that when the probability of the end of breakdown is zero, R_t will be equal to P_t (the cumulative probability that breakdown has occurred by time t) during the period. This provides a justification for the comparison in Figure 9.11.

There are two sources for the Breakdown Proportion R_t:

- observed data from the 64 day sample for the J8, J9 and J10 merge. These proportions will include breakdown due to blocking back.

- simulated data using the Aggregate Model to simulate the J8, J9 and J10 merges. These may or may not include blocking back depending on model status. The model run size for each day is 100.

These two sources of data are available for five separate days. Figures 9.12(a) and (b) show the comparison between the two series for Junction 8 and Junction 10 respectively

Apart from Monday, there is a close fit between the modelled and observed Breakdown Proportions in the AM peak.

The modelled Breakdown Proportions in the PM peak follows a similar profile to the observed, but there is a slight tendency for the modelled breakdown to be later than observed.

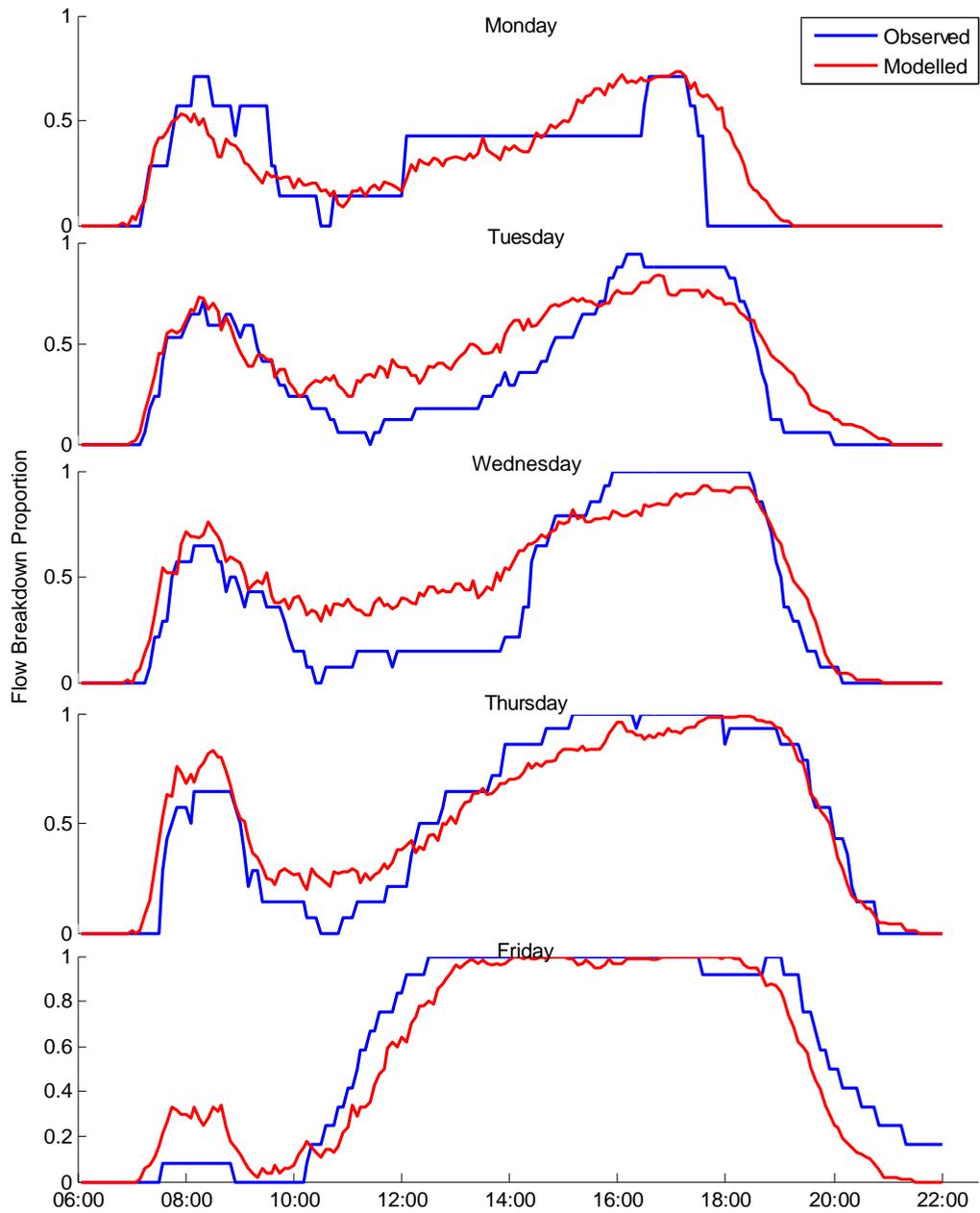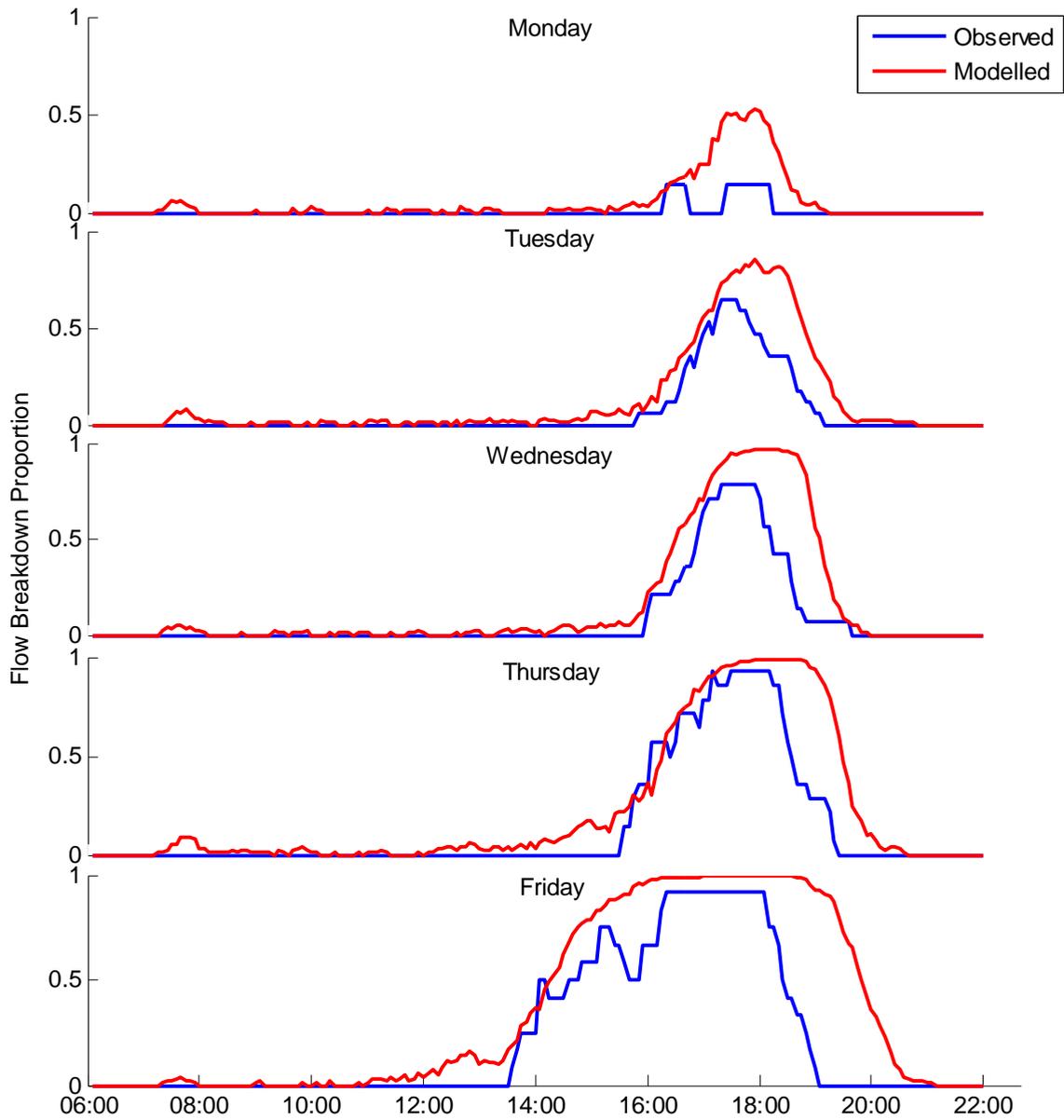## Figure 9.12(a): Breakdown Proportion by Day of Week (Junction 8)

Figure 9.12(b): Breakdown Proportion by Day of Week (Junction 10)

9.3.2   Travel Time – Observed and Simulated

Separately for each day of the week, Figures 9.13 (a) to (e) provide plots of the observed travel times for each day in the sample. Beneath the observed is the corresponding plot for a set of simulated days. The number of simulated days is the same as the observed
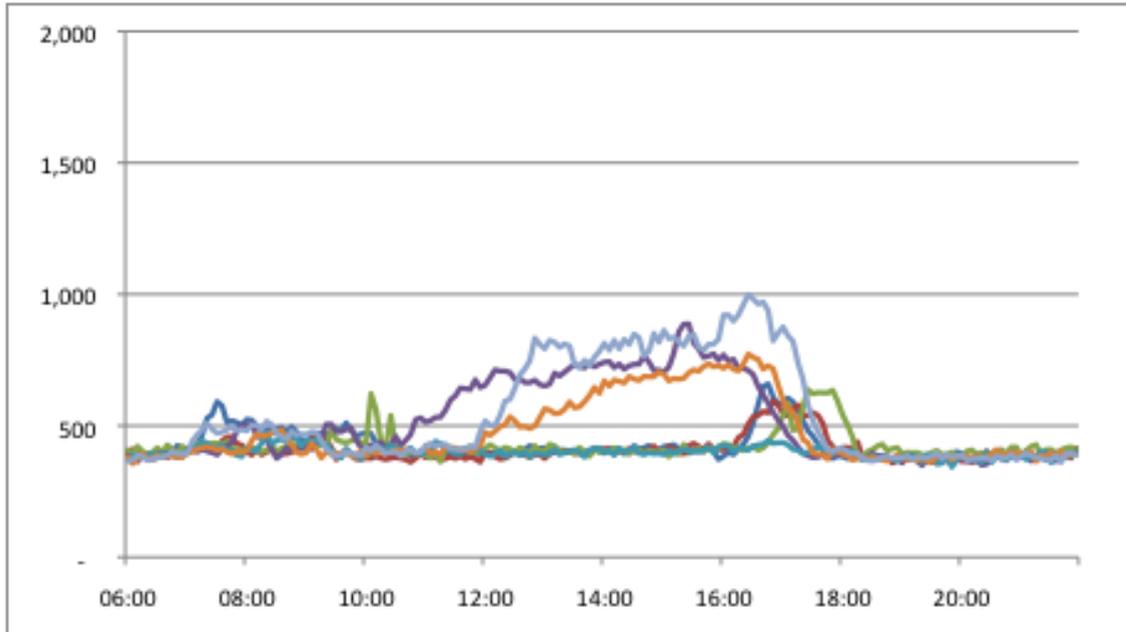
It can be seen that on Monday the incidence of flow breakdown is low, but the model does succeed in reproducing the general features of the variation. Tuesdays, Wednesdays and Thursdays have a generally similar pattern, but increasing in intensity of breakdown: again the model reproduces this well, though the level of variation is slightly understated.

As noted elsewhere, the pattern is different on Friday, with delays starting much earlier. This also is well reproduced by the model.

Hence, while we are not trying to replicate individual days, it appears that the model is generating simulated days which have similar outcomes in terms of the variation in travel time throughout the day, and the variation from day to day.

Motorway Travel Time Variability—Final Report
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Figure 9.13(a): Travel Time by Day of Week - Monday

Observed

Simulated

Motorway Travel Time Variability—Final Report
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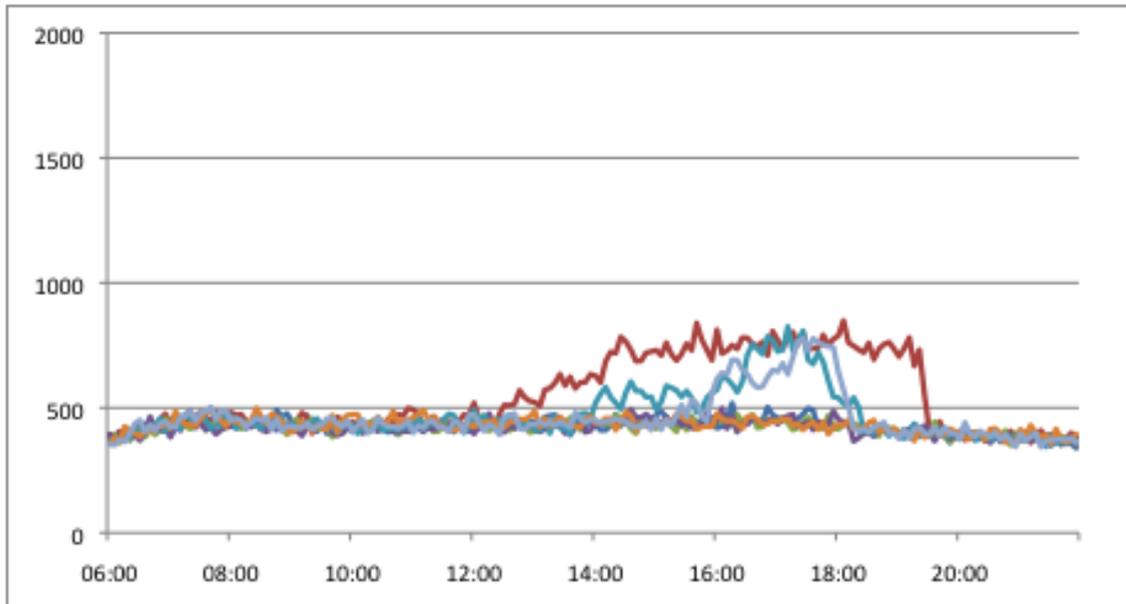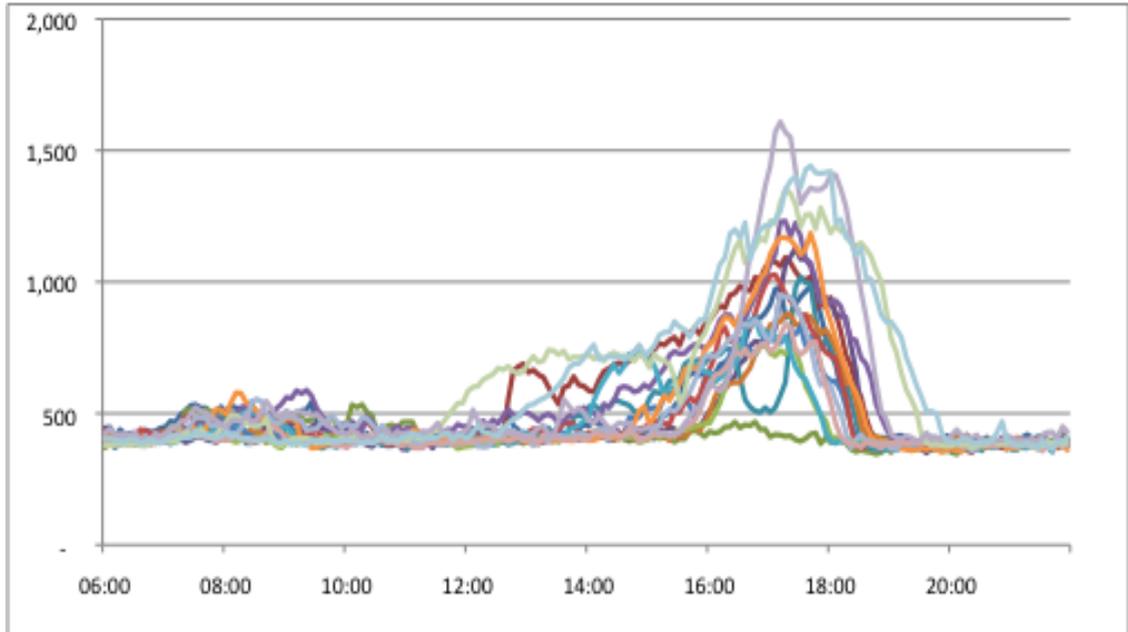
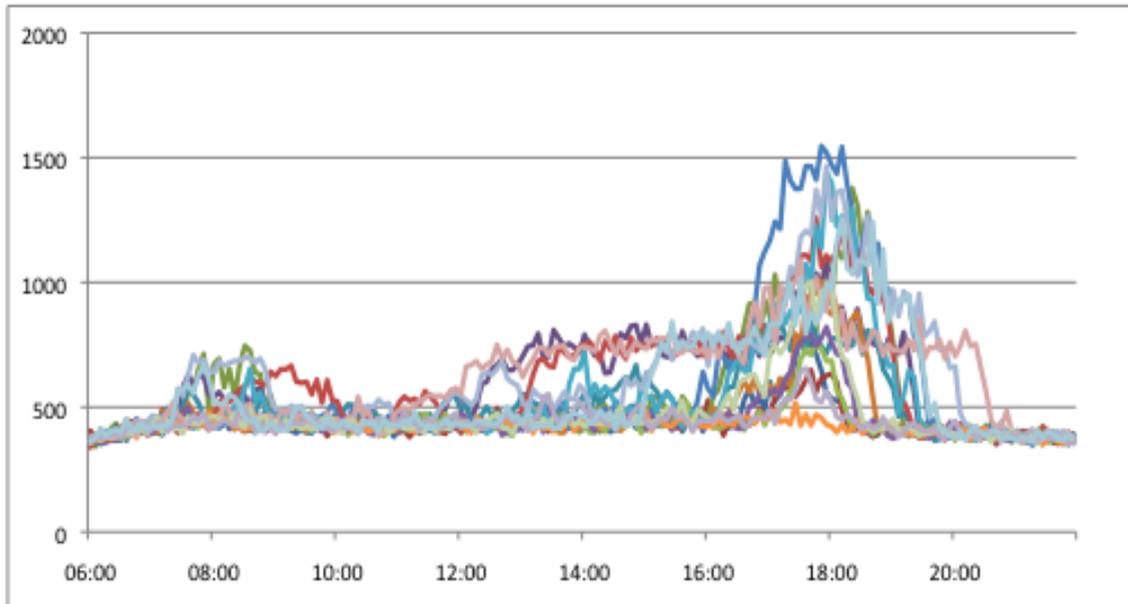**Figure 9.13(b): Travel Time by Day of Week - Tuesday**

Observed

Simulated

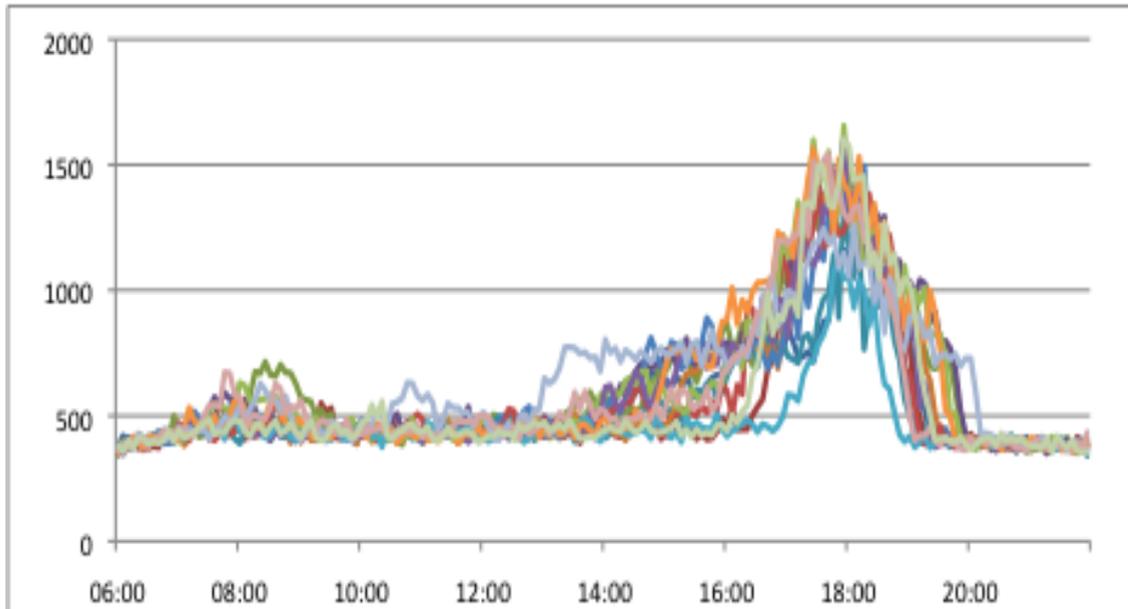**Figure 9.13(c): Travel Time by Day of Week - Wednesday**

Observed
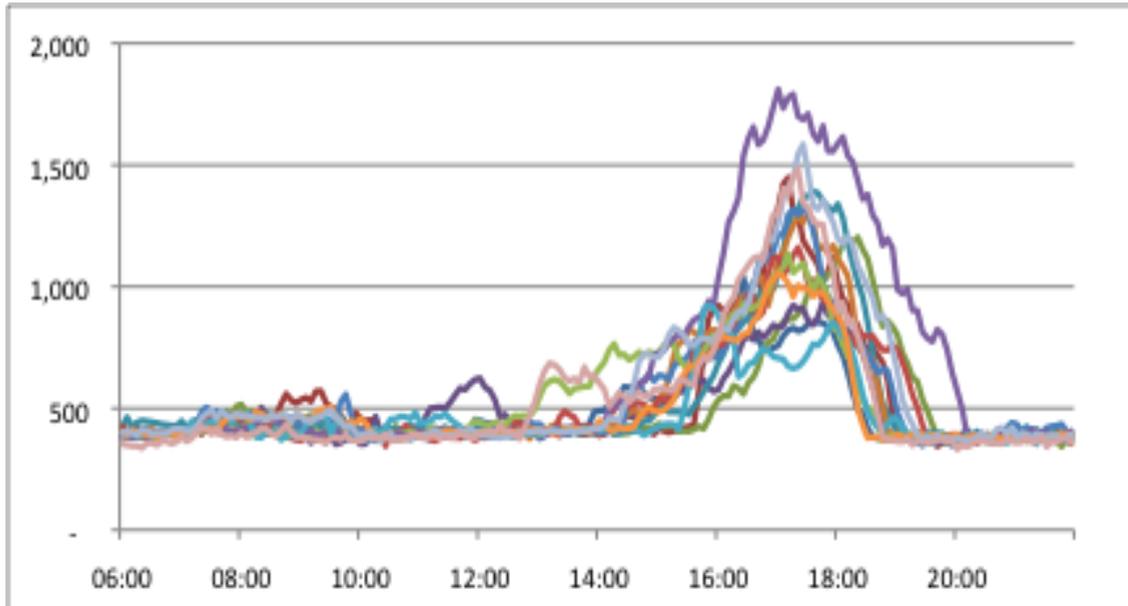
Simulated

Figure 9.13(d): Travel Time by Day of Week - Thursday

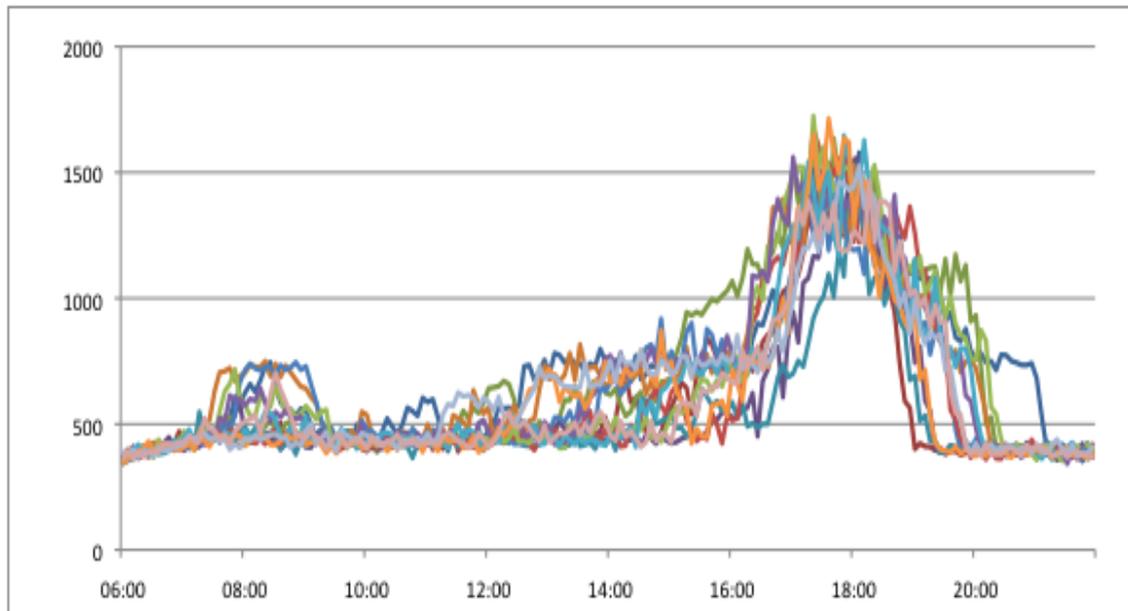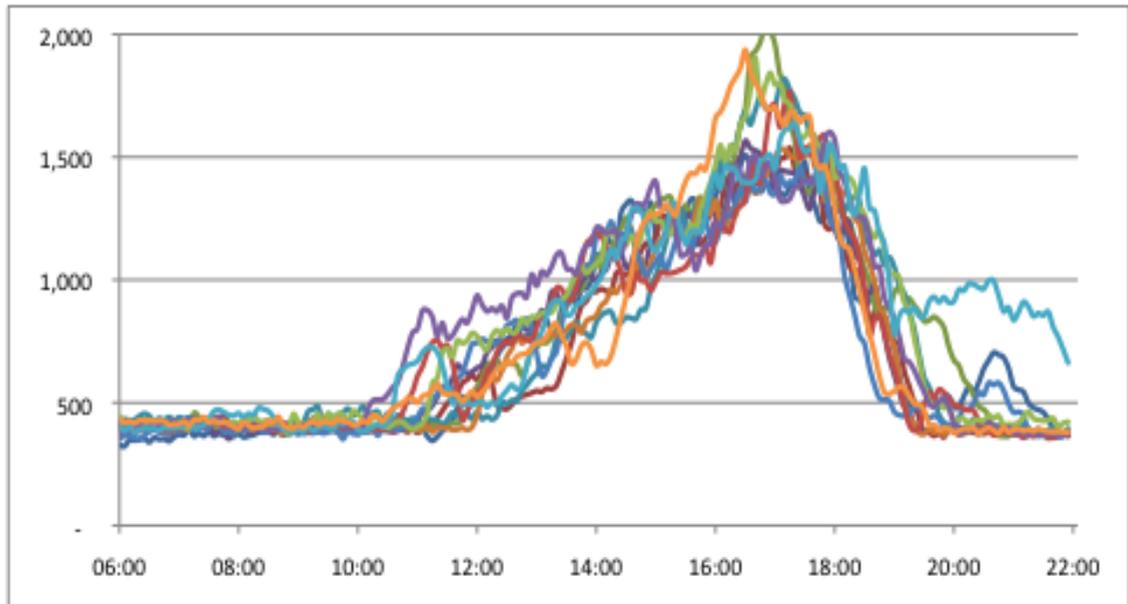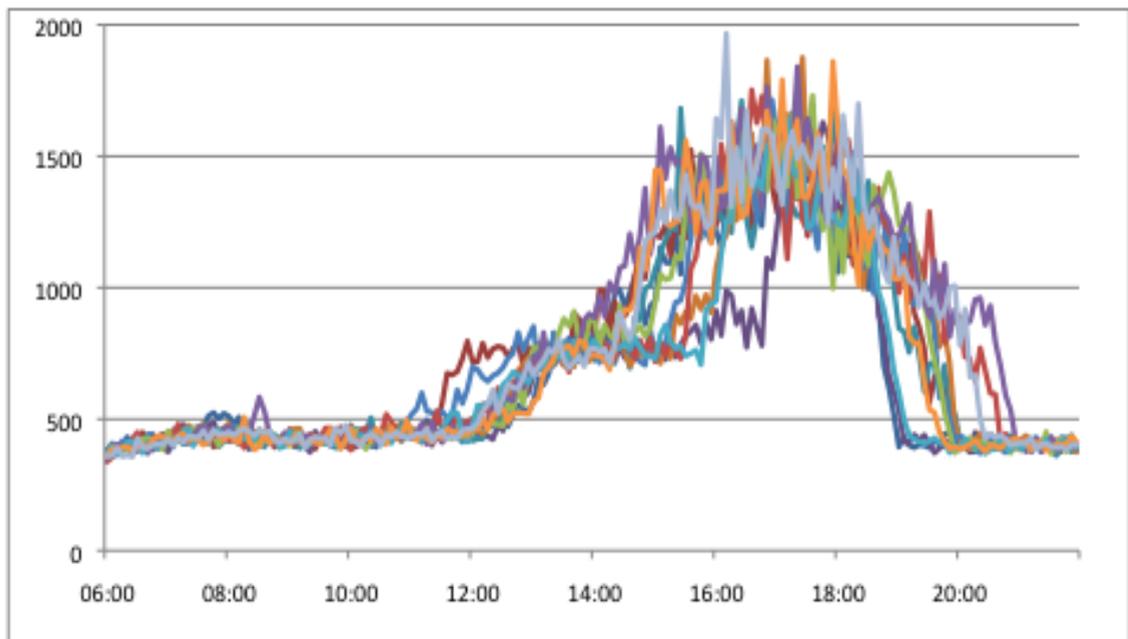**Observed**
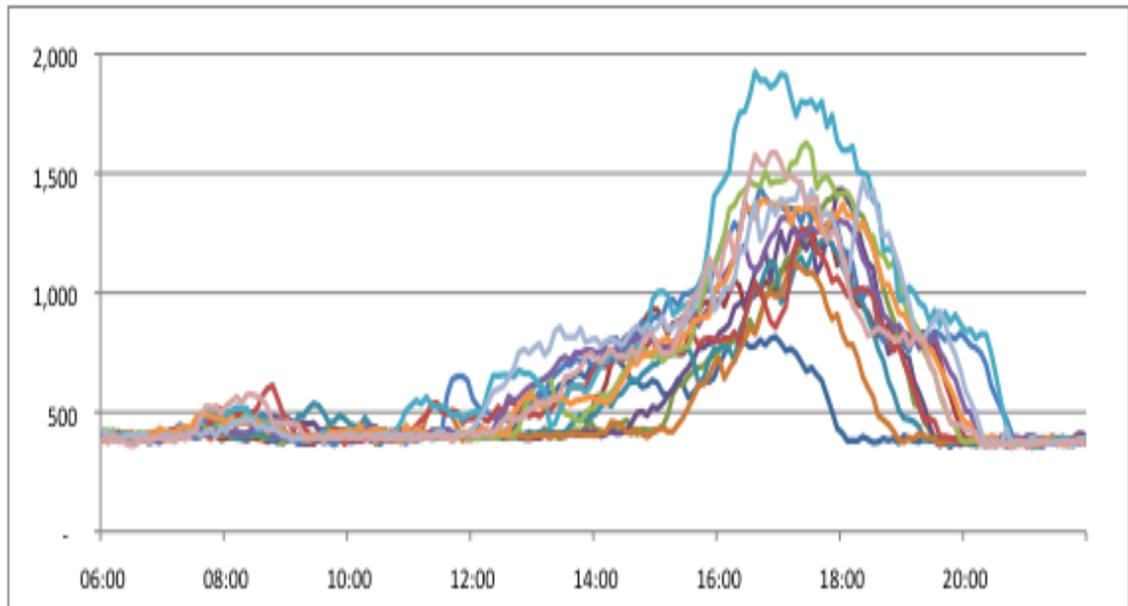
Simulated

Figure 9.13(e): Travel Time by Day of Week - Friday

Observed

Simulated

9.3.3   Modelling of Mean Travel Times

Figure 9.14 sets out the Aggregate Model estimates for the mean travel times, from camera 1a to camera 4, compared to the observed travel times for the 64 day sample.

The modelled travel time profiles reproduce the weekly pattern of Monday having the lowest maximum travel time, rising through the week with the highest on Friday. The modelled maximum travel times and travel time profiles are similar to the observed for all days. However the rise in travel time, and to a lesser extent its fall, is slightly later than observed. It should be noted that both Thursday and Friday are based on an interpolated demand profile. A better fit could be obtained by adjusting the assumed profile (maintaining the total demand constraint). However, the modelled travel time shown here is still based on the original interpolated demand profiles.

9.3.4   Modelling of TTV – Standard Deviation

The observed and modelled travel time variability (as measured by standard deviation) is shown in Figure 9.15. There is a close correspondence between observed and modelled on Tuesday, Wednesday and Thursday. The modelled values lie easily within their confidence intervals which is c.±50% depending on sample size. On the other hand, there is a large divergence between modelled and observed on Monday. Closer examination of the data shows that out of a sample of seven days, three exhibited very early breakdown and unusually low values of QDF for some periods. There is no record of any incidents or special events on these three days.

Friday also contains certain periods when there is obvious divergence between the two series. For most of the period the modelled Standard deviation lies within the 95% confidence intervals for the observed. The significant divergence around 1100 and again post 2100 are due in both cases to two (different) days where travel time was unusually high for no apparent reason. The low peak in observed standard deviation on Friday compared to other days is reproduced by the model and can be explained by the fact that breakdown times experienced on Friday are more concentrated than on other days of the week.

9.3.5   Modelling of TTV – Coefficient of Variation

The observed and modelled travel time variability (as measured by the coefficient of variation) is shown in Figure 9.16. The comments that apply to the travel time standard deviation results also apply for the coefficient of variation. The fit between observed and modelled is generally good except for Mondays (3 out of 7 days being exceptional) and two periods on Friday (1030-1130, post 2130) which are explained by two days in the sample showing unusual behaviour for short periods.

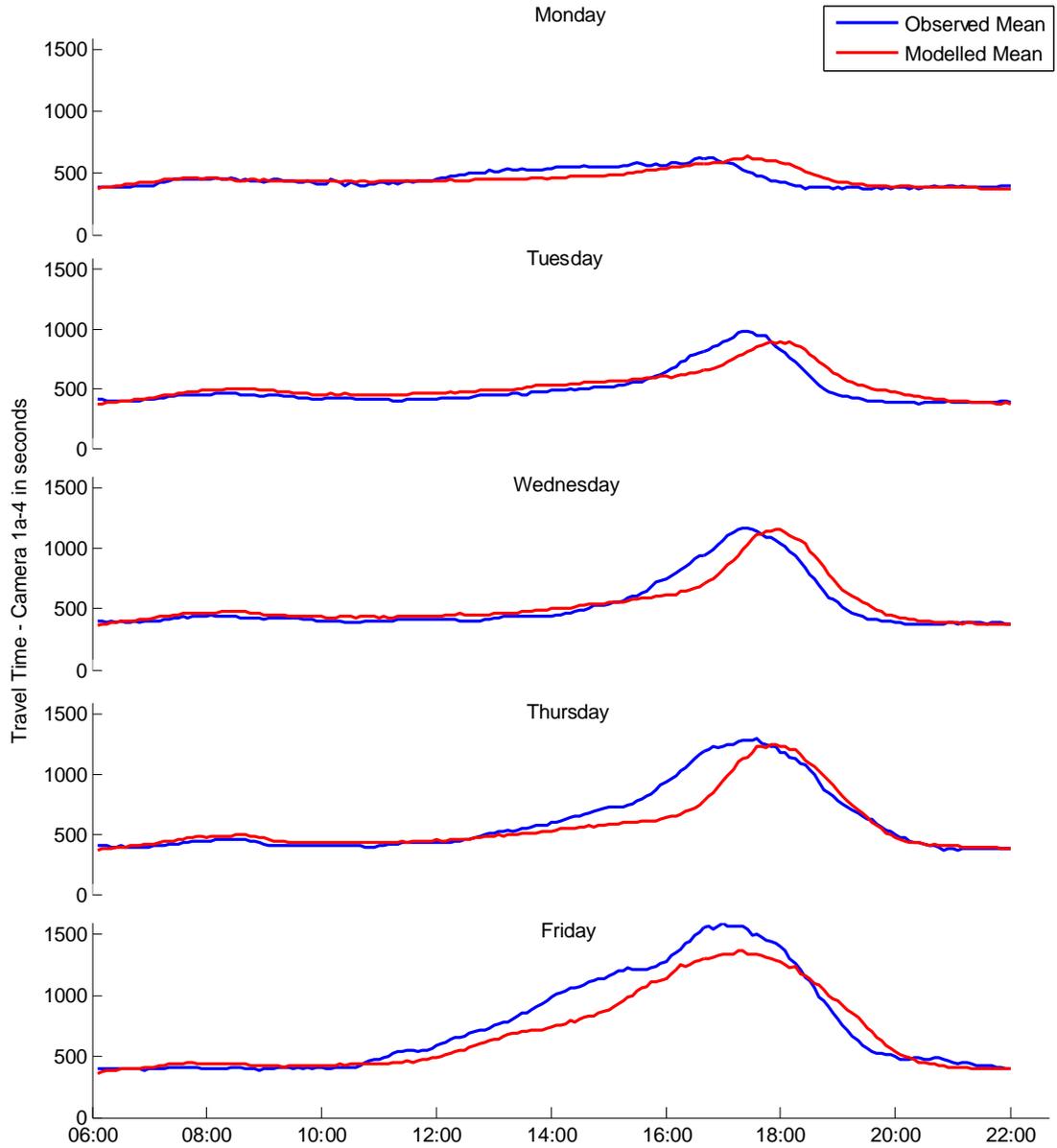# Figure 9.14: Mean Travel Time by Day of Week (Camera 1a-4)

Figure 9.15: Travel Time Standard Deviation by Day of Week (Camera 1a-4)

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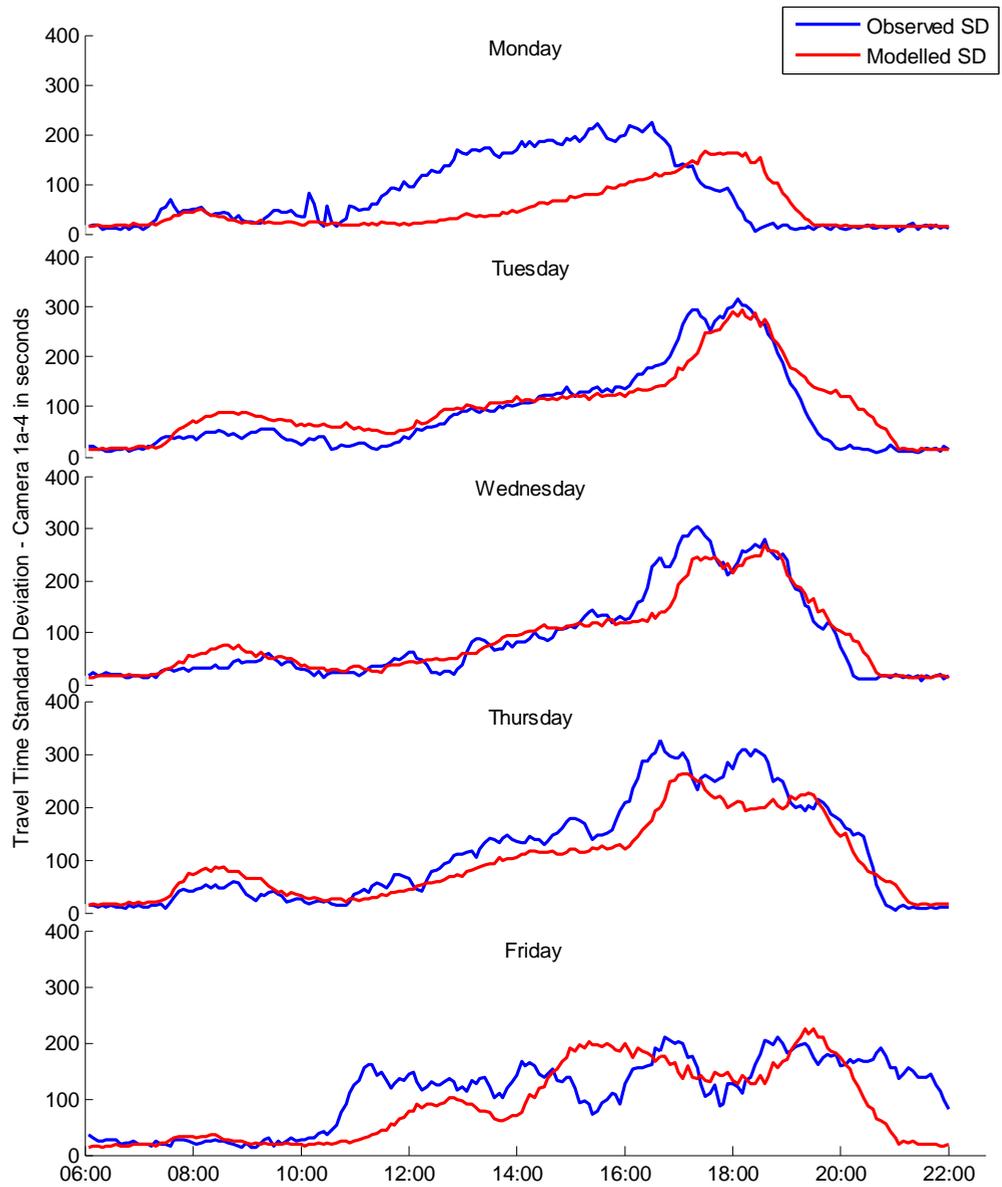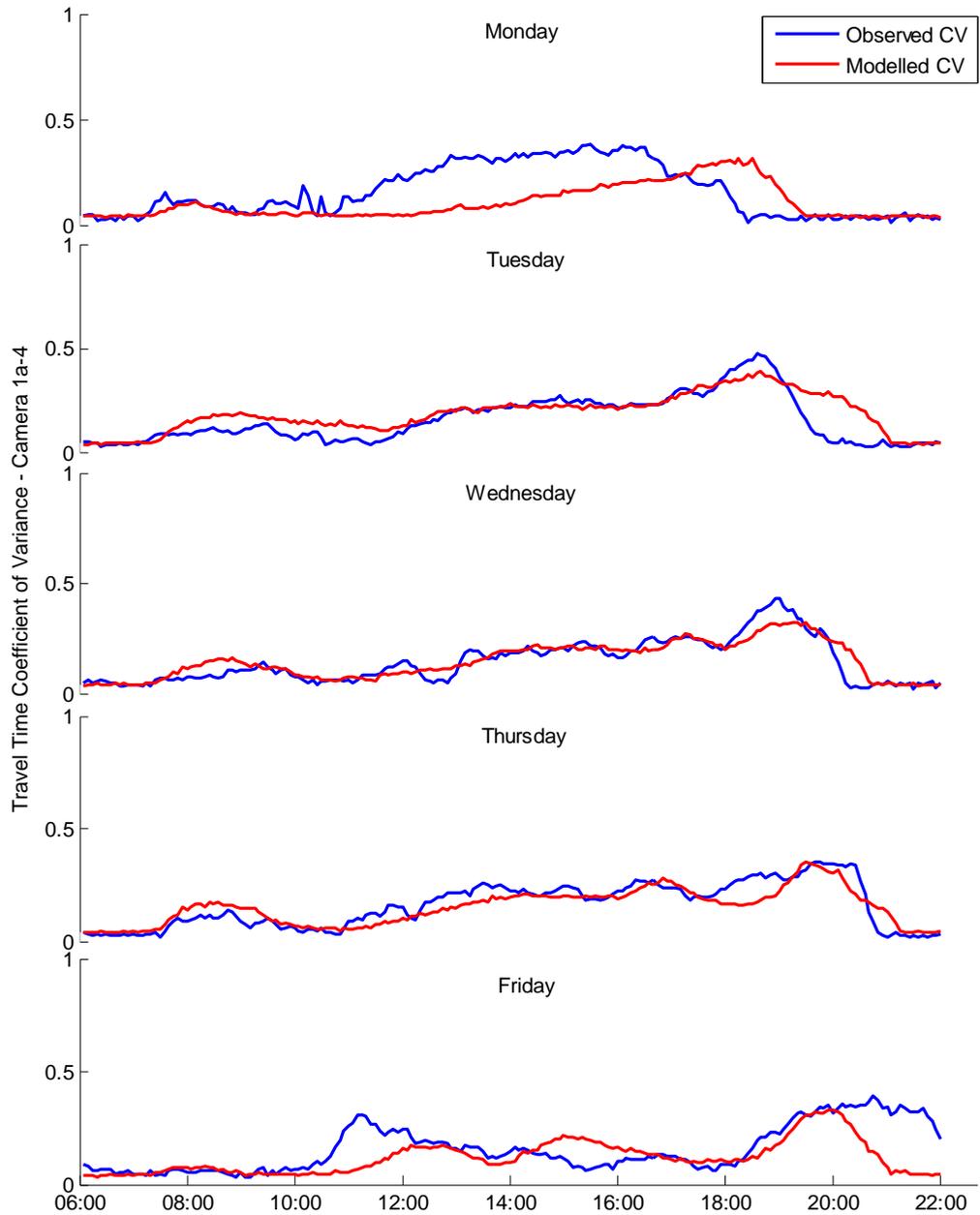
Page 106

Figure 9.16: Travel Time Coefficient of Variation by Day of Week (Camera 1a-4)

9.4   Conclusions on the Aggregate Model

The validation exercise has shown that the Aggregate Model can reproduce the phenomenon of breakdown and its effect on the profile of both average travel time and of TTV with a high degree of accuracy. In the case of mean travel time the difference between weekdays is reproduced well as is the pattern of travel time over the breakdown period. In the case of TTV (represented by standard deviation or coefficient of variation) both the pattern and scale of the change in TTV is reproduced.

There are four key components of the Aggregate Model:

• a simple dynamic assignment process;

• a set of stochastic demand profiles;

• a breakdown function (BDF) for each merge; and

• a queue discharge function (QDF) for each merge.

A good match between modelled and observed travel time and its variability means that the various model components are also performing well. An important component for accurate prediction is an accurate demand profile. The greater delay found on Friday is due, not to greater total demand, but rather the different demand profile found with demand approaching breakdown levels much earlier in the day at junction 8. Accurate daily demand profiles (which differ by day and by entry/exit in this study) are essential if the travel time profile is to be reproduced with any accuracy The BDF and QDF functions were calibrated on days which included the set of days that were used for model validation. Close examination of the breakdown pattern and duration of breakdown in the modelling phase for each day of the week confirmed their validity.

The importance of an accurate estimate of day to day variation in demand should also be emphasized. TTV is very sensitive to this parameter.

The Aggregate Model is a simple model based on a minimalist definition of the road network and a few simple assumptions. Its ability to reproduce daily profiles of both average travel time and TTV at the level of accuracy seen in this exercise using data from the M6 suggests it has a useful role to play in the analysis of breakdown and can also provide a useful predictive tool.

10 CONCLUSIONS AND RECOMMENDATIONS

10.1 Summary of Results

Analysis of the survey data for M6 Junctions 8 to 10A shows that on all weekdays except Mondays, journey times and TTV increase dramatically in the afternoon due to flow breakdown.

Two types of flow breakdown were identified: 'shockwaves'; and 'standing waves'. The 'standing waves' which occur at the Junction 8, 9 and 10 merges are the main cause of increased travel times. Conversely 'shockwaves' which start within links have relatively little effect. In consequence it is concluded that to forecast the mean travel time and TTV effects of flow breakdown, it is necessary to accurately model the 'standing wave' flow breakdown.

Delay and TTV caused by 'standing wave' flow breakdown in the case of M6 is dictated more by supply than demand variability. In particular, the time at which flow breakdown occurs is critical in determining the level of delay and TTV. Even though the underlying probability of breakdown in any 5-minute period is low, within a prolonged period of high demand breakdown at some time becomes almost certain.

An "aggregate" model has been developed in order to forecast flow breakdown and its mean travel time and TTV effects. The model adopts a simulation approach at a relatively aggregate level, based on queueing theory for periods where 'standing wave' flow breakdown occurs and on speed flow relationships for other periods. It models merges and the links between them, and operates on five minute time periods throughout the day, i.e. 0600-2200.

The Aggregate Model has four main components:

- the (stochastic) profile of demand for traffic entering the modelled network;
- a simple dynamic assignment model, in which the network is represented by a relatively small number of links;
- a "Breakdown Probability Function", which forecasts the probability of flow breakdown occurring at a given time; and
- a (stochastic) "Queue Discharge Function", which forecasts the rate at which the queue discharges once flow breakdown has occurred.

Stochastic supply functions in the model forecast a) the probability of flow breakdown at particular flow levels and b) the reduced capacity at queueing locations. Stochastic demand functions incorporate mean demand profiles from 0600-2200, by five minute period, for each day type and for each entry (M6 main line and on slips), as well as day to day variability in total demand and the variation from the mean found in five minute periods.

To represent the variability in the supply and demand functions forecasts are produced using a process based on 100 simulated days for the supply and demand parameter values. All model parameters were derived from observed data for M6 Junctions 8 to 10A.

Validation of the Aggregate Model shows that it can reproduce the observed pattern of flow breakdown. More critically, validation against independently observed ANPR journey time data shows it can reproduce the general profile of both journey times and TTV throughout the day for most day types.

Finally, we should note that the results of this work vindicate the decision taken in 2003 to collect a detailed data set over a year for a small section of congested motorway. Although some lessons have been learnt as to the difficulty of achieving complete consistency, especially in the face of random instrument failure, the outcome data set is an important product of the research. It has been carefully described and archived, and could be used for further analysis. As far as we are aware, such a data set is unique, and certainly of value in its own right.

10.2 Discussion of More General Application

10.2.1 Overview

The research has demonstrated the importance of 'standing wave' flow breakdown in relation to journey times and TTV on congested sections of the motorway network (and by implication congested sections of the highway network in general). These issues are not adequately addressed by current transportation techniques for operational and economic appraisal. This is a major omission from appraisal which is likely to lead to the mis-estimation of benefits for schemes relating to congested sections of the highway network where flow breakdown occurs, and consequent mis-specification of scheme features and priorities for congested links.

The Highways Agency is required to meet targets for improving journey time reliability on the motorway and trunk road network. The analyses set out in Sections 3.6 and 4.4 suggest that incidents and 'standing wave' flow breakdown account for the vast majority of TTV on the network. However, there is no methodology for forecasting TTV arising from 'standing wave' flow breakdown.

WebTAG Unit 3.5.7 "The Reliability Sub-Objective" sets out the DfT's currently recommended approach to the appraisal of TTV. It notes that for inter-urban roads when flows are below capacity, TTV arises mainly from incidents, and for appraisal of TTV in this situation INCA is the DfT's recommended approach. However, the case where the flow exceeds capacity and flow breakdown occurs is not dealt with by INCA as it is only applicable when flows are less than or equal to 95% of capacity. Thus there is no current mechanism for assessing TTV in these circumstances.

In the wider context of appraisal, the situation is worse than this implies, because – potentially more importantly – there are no adequate techniques for forecasting the excess journey times caused by flow breakdown.

In the light of the conclusion that the Aggregate Model approach is the way forward for the modelling of travel time and TTV due to flow breakdown, an appropriate implementation of the Aggregate Model would remedy these shortcomings.

The Aggregate Model developed in this research is specific to M6 Junction 8 to 10A and is based on MATLAB software. It will be necessary to consider how the model should be developed to allow more generalised application in support of highway management and scheme development.

With further analysis the approach could be extended to cover different types of 'standing wave' flow breakdown, both on motorways and other roads. This would provide much better estimates of the benefits of measures to relieve flow breakdown than can be provided by current transport modelling techniques. It would be able to address measures ranging from Managed Motorway strategies to major capacity enhancements such as widening.

Appraisal of travel time benefits is usually modelled by forecasting impacts for fixed periods, e.g. AM peak, PM peak, inter-peak, which can then be extrapolated to give annual totals. However, this approach is problematic for modelling the impacts of 'standing wave' flow breakdown: a) because travel time and TTV effects in any particular time period are often dependent on supply and demand in previous periods; and b) as we have shown, because the actual shape of the profile can be critical to the outcome. At the merges covered in this research, flow breakdown commences in the inter-peak, therefore the travel time and TTV in the PM peak is dependent on the size of flow breakdown queues at the end of the inter-peak, in addition to the PM peak demand.

In view of the above, we believe that the best methodology for modelling the effects of 'standing wave' flow breakdown would be to apply the Aggregate Model approach over the whole day, generally taken as 16 hours between 0600 and 2200. For local operational and economic appraisal such models would be similar in scale to the Aggregate Model developed for this study. However, for the appraisal of larger schemes, the technique would need to be applied to a wider network and to deal with how traffic routes through the network. This might be through interaction with a strategic assignment model such as SATURN. It would probably also be necessary to consider demand variability on a route basis rather than on an entry/exit basis. These and other issues relating to implementation for a wider network require further consideration.

To estimate **overall** TTV it would be necessary to include incident related TTV in the Aggregate Model. It would not be practical to do this on a stochastic basis within the current application which uses 100 simulated days to generate the variability, because incident occurrence frequencies are low and their locations, timing and supply characteristics are highly variable. Nonetheless, an integrated procedure has considerable attraction, especially given that both the Aggregate Model and INCA make use of queueing theory methodologies. Further, the use of demand profiles rather than the constant demand in each standard COBA "flow group" used in INCA would potentially improve the accuracy of incident delay and TTV estimation. In reality flows vary during the time the incident reduces capacity. INCA assumes that over all flow groups this averages out, but this may not be the case. Use of demand profiles would also allow the effects of incidents occurring in periods where flows exceed 95% of capacity to be assessed which would address a major weakness in the current INCA methodology.

There are thus a number of possible options for generalised application, relating both to the short term and longer terms. Essentially there are three interconnected aspects which need to be considered:

§ the type of software to be used for the implementation;

§ the "coverage" of the implementation, in terms of road types and junction types;

§ whether to incorporate the treatment of incidents within the implementation

We now consider how each of these aspects might be handled in the short and longer terms.

10.2.2 Software

If the MATLAB platform is to be further used, some appropriate generalised interface for input and output will be required, particularly to allow the appropriate links to be defined. In the slightly longer term, if the approach was considered worth using for more general appraisal, purpose written software could be developed: this would require an agreed specification.

Since appraisal normally relates to future years, some process for adjusting demand parameters in response to forecast growth and/or for the do something case would be required.

For wider application, integration with network models would be needed. Given the importance of the temporal profile, this would require a fully dynamic assignment process which assigns traffic in small packets through the network taking account of time dependent delays. It would therefore be necessary to conduct a thorough review of dynamic assignment software packages.

10.2.3 Coverage

The current research has only addressed flow breakdown relating to three merges on the M6 within the West Midlands conurbation. For wider application of the approach it is necessary to establish the following for other 'standing wave' flow breakdown locations:

- supply parameters BDF, QDF, and block back parameters; and

- demand model relationships both daily and by five minute periods.

'Standing wave' flow breakdown occurs at bottlenecks on the highway network. For motorways these will include:

- merges at junctions with no lane drop, as for the M6 application;

- merges at junctions with a lane drop through the junction;

- diverges at junctions with a lane drop through the junction;

- diverges where traffic queues back from the diverge onto the main lane carriageway thus effectively reducing the carriageway width; and

- locations where there is a large reduction in effective capacity such as reductions in the number of lanes, steep gradients or toll booths.

For segregated trunk roads, bottlenecks will additionally include dual/single carriageway interfaces. For non-segregated roads they would additionally include junctions.

It is likely that the supply model parameters would vary significantly between these different bottleneck types. It is also likely that they would vary in relation to other factors such as:

- the detailed layout of the bottleneck, e.g. the length of merge lanes;

- the proportions of main line and merging/diverging traffic;

- the vehicle mix, particularly the proportion of heavy vehicles;

- the number of main line lanes (even after adjusting the parameters on a per lane basis); and

- whether speed/flow management systems are present such as Managed Motorway or permanent speed limits.

To apply the Aggregate Model on a wider basis it would therefore be necessary to establish BDF, QDF and block back parameters to cover all bottleneck types for a range of other factors which might significantly affect these parameters. There is thus a general question as to how far down the list of road types it is considered necessary to go.

On the demand side, the model requires 16 hour demand, five minute demand profiles and CVs for these for each entry to the system which is being modelled for each day grouping to be modelled. The parameters developed in this study relate a to a motorway which has a high proportion of long distance traffic but which is located within a conurbation in a direction outbound from the conurbation centre. For wider application it would be necessary to develop parameters for different types of location.

The daily demand will always be entry specific. However, day groupings, five minute demand profiles and CVs will be related to the location type and traffic mix.

Location types which affect these parameters would include:

- whether inbound or outbound to/from the regional centre;

- area type, e.g. conurbation, inter-urban, holiday area;

- location relative to major traffic generators, e.g. distance from a regional centre would affect the timing and intensity of commuting peaks; and

- entry type, e.g. main line motorway or slip road.

Traffic mix type would depend on the relative importance of trip vehicle type and purpose proportions, particularly:

- commuting and education trips which are mainly concentrated in the AM and PM peaks;

- social, recreational, shopping and personal business trips which tend to avoid the peaks, although they can be a significant element in the PM peak;

- employers' business trips which are generally spread over the working day;

- holiday trips which mainly occur around weekends;

- light goods vehicles trips which are mainly spread over the working day; and

- heavy goods vehicle trips spread over the working day and for trunk haulage and some distribution the night, but tending to avoid the peaks.

For wider application of the model it may be possible to develop relationships based on entry location types and their traffic mix. This would avoid the need for developing observed relationships for each entry.

In developing these relationships, particular account would need to be taken of the throttling of demand due to 'standing wave' flow breakdown. Ideally the entries used for parameter development would be located beyond the maximum extent of the queueing to allow actual demand to be measured.

For both the supply and demand functions, detailed traffic count and speed data can be provided by MIDAS for most motorways where 'standing wave' flow breakdown is likely to occur. Given the MIDAS data, the required inputs could be obtained straightforwardly using the type of analysis described in Chapters 7 and 8. For all purpose roads it would be necessary to undertake specific data collection which would be costly and time consuming. In view of this it might be appropriate to concentrate on motorways in the first instance.

10.2.4 Incidents

The current recommended method INCA operates using average incidents for each incident type with varying impacts in terms of duration and percentage of capacity lost. As noted, the demand level (within any COBA flow-group) is assumed to be constant. The mean impact of different types of incident, in terms of delay and TTV, are estimated, and then correction factors are applied to allow for the distribution of different incident characteristics.

It would be possible to use the Aggregate Model, with its temporal profiles over the whole day, to establish what the impact of the constant demand assumption is, and whether it is possible to adjust the INCA parameters to take account of it. To achieve this it would be necessary to undertake simulations for each individual incident in the incident database based on a range of flow profiles for 5 minute time periods. The delay and TTV derived from the simulation would then be compared with that forecast by INCA.

Our expectation is that the use of detailed profiles will produce substantially different outcomes in terms of the incident-related delays and TTV, for the same reasons that apply in the case of breakdown – i.e., that the actual time at which the queueing begins relative to the demand profile has a major impact on the variance.

This approach could also be used to assess the effect of INCA not forecasting the effects of incidents where flows exceed 95% of capacity.

10.2.5 Validation

Finally, we need to consider the requirements for validation. Ideally the Aggregate Model should be validated, in terms of modelled journey time and TTV profiles, for the full range of supply and demand types to which it is to be applied. However, it may be adequate to do this for a **sample** of supply and demand characteristic types, and in the shorter term, with a view to making progress, it might be considered acceptable to rely on the validation within the current project.

For validation detailed journey time data is required. For MIDAS-equipped sections of motorway HATRIS produces journey times based on an algorithm which uses the MIDAS spot speeds. However, there are some concerns about the accuracy of the output in locations where significant queueing occurs. Hence, while HATRIS data would be the most cost-effective source, further examination of its reliability in the particular circumstances of flow breakdown will be required.

If, as we suspect, the HATRIS data turns out not to be appropriate for validation, it will probably be necessary to install ANPR cameras to derive observed route journey time and TTV data. However, this may only need to be done for a limited number of locations – preferably those where motorway management or improvement schemes are being considered as the ANPR data would also be useful in scheme development. Validation would then be carried out as in this study.

In addition to the foregoing, which relates primarily to the use of TTV and journey time in scheme appraisal, consideration also needs to be given as to how the findings could be used to inform motorway management and operational analysis to minimise delay and TTV caused by flow breakdown. For example the control systems for ramp metering and IDM could be amended to reduce the probability of flow breakdown thus delaying its onset or avoiding it completely and to induce recovery by restraining flows below QDF. This would use current MIDAS counts plus short term forecasts to cover the lag period between demand entering the system and reaching the flow breakdown location.

10.3 Recommendations for Future Work

10.3.1 Overview

On the basis of the foregoing, we set out some recommendations for taking the conclusions forward. As the discussion above indicates, there are options relating both to the short term and the longer term. By the short term, we generally have in mind what might be achieved within the current financial year.

10.3.2 Development Strategy

To take the work forward with a view to rectifying the shortcomings in current techniques which we have identified, it would be necessary to give consideration to the **strategy** for implementing the Aggregate Model in general scheme appraisal, in the context of current approaches to the modelling and appraisal of highway schemes and other interventions. It would be necessary to address both schemes with localised effects and those with wider effects. In addition, the three key aspects identified (software, coverage and the integration of incidents) would form a major part of the strategy. Under the heading of "coverage" there would need to be an appropriate plan for the derivation of supply and demand parameters, and the needs for validation would also be agreed.

While we feel that considerable progress could be made in the short term, there would clearly be a need for extensive discussion both with the HA and the DfT, and this might realistically require a slightly longer timescale.

We recommend that in the short term an initial strategy should be developed to identify options and their implications and to inform this discussion. At some stage, it will be necessary to decide on the appropriate software platform for wider implementation. With this in mind, we also propose to review of current "dynamic assignment" packages. Significant progress could be made on this within the short term

There are other useful short term developments noted below which could be made without having a definitive version of the strategy.

10.3.3 Model Implementation

Given the need to manage the congested sections of the motorway network, it would be advisable in the short term to focus on the main types of location where 'standing wave' flow breakdown occurs on the motorway network. The model could readily be adapted to other small sections of motorway. While the supply and/or demand parameters in the model would need to be modified to reflect the particular characteristics of these sections, the required data should generally be readily available for sections which are equipped with MIDAS. These are likely to cover most motorways where 'standing wave' flow breakdown occurs. It is understood that this data is readily available for a number of recent years.

In the short term, we might be willing to assume that the dynamic assignment component of the model functions correctly, so that transferring to other motorway sections is primarily an input data issue. It should be assumed that in all cases the relevant MIDAS data will need to be available, and analysed appropriately. There would be some advantage in first testing the approach in new locations which have supply and demand characteristics which differ from the M6 Junction 8 to 10A.

The first stage in identifying suitable motorway sections would be to examine MIDAS MTV plots to identify locations where 'standing wave' flow breakdown occurs frequently and to establish MIDAS availability for these. Parameters could then be estimated for a range of supply and demand types for which appropriate MIDAS data is available. It would be beneficial to focus on locations where the Highways Agency are considering interventions, which would yield specific parameters for the appraisal of these.

It would also be of interest to compare the model predictions of average journey time over a suitable period (e.g. COBA flow group) with what would be obtained from a more conventional "time-aggregate" assignment model (such as SATURN), with a view to understanding how serious the effects of not taking a dynamic approach are.

Large amounts of HATRIS data are readily available in 15 minute periods. Consideration could be given to whether this data could be used to make appropriate predictions of travel time and TTV.

Ideally the operation of the Aggregate Model should be validated for these different parameters and for different network configurations. Whether this is feasible in the short term depends on whether HATRIS data can be used for this purpose, or whether ANPR is required.

In the somewhat longer term it would also be advisable to address other road types which have 'standing wave' flow breakdown, e.g. dual/single interfaces on all purpose roads. This would allow the model to address the full range of flow breakdown locations and demand profiles. For other roads which are not MIDAS equipped it would be necessary to collect new data. This would therefore be a longer term process than the coverage of motorways.

10.3.4 Integration of Incident Methodology

Whether or not a decision is made to move to a combined treatment for incident- and flow breakdown-related TTV, it would be advantageous to investigate the performance of the current INCA methodology within the dynamic assignment represented by the Aggregate Model. Given our detailed understanding of the INCA methodology and the simulation with individual incidents to establish the distributional effects of incident characteristics (both developed by John Fearon Consultancy), this could be achieved within the short term. It would assist with the recommendation as to whether to develop the two kinds of TTV (and journey time) methodologies separately or together. It would also indicate whether the existing INCA methodology is appropriate in terms of its flow group assumptions and its capping of flows at 95% of capacity.

10.3.5 Motorway Operational Analysis

This study has provided an understanding of the mechanisms which cause 'standing wave' flow breakdown, and this could inform the HA's operational analysis.

In the short term we recommend that current motorway management control systems would be reviewed in the context of this study. Enhancements would be identified which would improve the management of 'standing wave' flow breakdown. Consideration would be given to how this could be extended to include incidents, although these present the difficulty that the loss of capacity (i.e. the lane closure time profile) is generally difficult to predict.

Current management systems such as the HA's 'reliability management delivery plan' and the new journey time reliability targets would be reviewed. Consideration would be given as to how the approaches developed in the study should be used to support these systems.

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