

# Managed Motorway Design Guide

## Volume 1: Role, Traffic Theory & Science for Optimisation

### Part 3: Motorway Capacity Guide





## Managed Motorway Design Guide

### Volume 1: Managed Motorway Role, Traffic Theory and Science for Optimisation

#### Part 3: Motorway Capacity Guide



#### Volume 1 - Managed Motorways - Role, Traffic Theory and Science

- Part 1 - Introduction to Managing Urban Motorways
- Part 2 - Traffic Theory Relating to Urban Motorways
- **Part 3 - Motorway Capacity Guide**
- Part 4 - Road Safety on Urban Motorways
- Part 5 - Linking Investment and Benefits Approach



#### Volume 2 - Managed Motorways Design Practice

- Part 1 - Managed Motorway - Design Principles and Warrants
- Part 2 - Managed Motorway - Network Optimisation Tools
- Part 3 - Motorway Planning and Design
- *Part 4 - LUMS, VSL, Traveler Information (Update Under Development)*



# Volume 1: Managed Motorway Role, Traffic Theory and Science for Optimisation

## Part 3: Motorway Capacity Guide

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“The world we have created is a process of our thinking.  
It cannot be changed without changing our thinking”

Albert Einstein

## Preface

The process of evaluating and comparing the operational effects of alternative road planning and design scenarios is at the core of different Highway Capacity Manuals (HCM) around the world. It allows analysts to screen a variety of scenarios and select a reasonable one to ensure optimal investment in infrastructure. Central to this process are capacity values or Maximum Sustainable Flow Rates (MSFR) for certain road infrastructure specifications (e.g. number of lanes) which if compared to the forecast traffic demand allow for a prediction of the expected traffic flow quality (or Level of Service).

For more than 60 years, the US HCM has been the model for comparable guidelines in other countries. While most of the traditional HCM values and procedures revolve around the 'fundamental diagram' which links the three basic parameters of speed, flow rate (volume) and density, this does not allow for a transparent selection of an appropriate maximum sustainable flow rate. However, this approach is now evolving internationally: Inspired by contemporary traffic flow theory, the new US HCM 2016 as well as the Dutch HCM foreshadows an approach which puts the probability of flow breakdown at the centre of such considerations.

Since the establishment of its first Managed Motorway on the M1 Corridor between 2007 and 2010, VicRoads has placed a lot of emphasis on establishing and maintaining relationships with leading road traffic researchers around the world to develop detailed understanding of contemporary motorway traffic flow theory. Combined with the availability of very detailed and high-quality traffic data sourced from Victorian motorways, this allowed for a thorough assessment of the different approaches to defining capacity values and maximum sustainable flow rates in the various countries, with a particular focus on achieving the expected outcomes for drivers and passengers. As an extension of this work and as discussed also in Volume 1 Part 2 this approach has proven effective in real time operations, where the control targets for every motorway segment are 'limited to' (e.g. maximum setpoints) the occupancy values associated with each road segment's MSFR.

The need for thorough understanding of this complex subject matter and acknowledgement of the different needs of Guide users has led to separating this Guide into the following parts:

**Chapter 1** explains the classical understanding of traffic flow and operational performance and establishes the need for a refined approach in light of customer expectations, road operator's targets and recent research results. This chapter explains the principles.

**Chapter 2** provides guidance on the Maximum Sustainable Flow Rates for planning and design of motorways with a gradient of up to 5% and a Heavy Goods Vehicle (HGV) share of up to 30% (speed limit of 100 km/h or 80 km/h). This chapter provides values resulting from application of the science.

**Chapter 3** contains details on the 'capacity', productivity and flow breakdown probability determination methodologies including on the corresponding measurement results and other scientific considerations. This chapter provides the science/methodology used to determine values in Chapter 2.

## Table of Contents

<b>Preface</b>	<b>8</b>
<b>List of Figures</b>	<b>11</b>
<b>List of Tables</b>	<b>13</b>
<b>1 Assessing Motorway Section Operation</b>	<b>17</b>
1.1 Customer expectations and road operator's targets	17
1.2 Traffic flow and operational performance	17
1.3 Data and observations	19
1.3.1 International perspective	19
1.3.2 VicRoads perspective	19
1.4 Maximum Sustainable Flow Rate	21
1.5 Factors affecting Maximum Sustainable Flow Rates	21
1.6 Detailed description of Key Performance Indicators and their relationship	22
1.7 Concluding remarks	24
1.8 LOS density bands and factors influencing traffic flow	24
<b>2 Maximum Sustainable Flow Rates</b>	<b>29</b>
2.1 Why are Maximum Sustainable Flow Rates needed?	29
2.2 Maximum Sustainable Flow Rates for managed motorways	29
2.2.1 Carriageways	29
2.3 Corridors	32
2.4 Tunnel sections	32
2.5 Maximum Sustainable Flow Rates for unmanaged motorways	35
2.6 Typical speed-flow relationship curves	39
2.7 Auxiliary lanes	41
2.8 Tight curves	42
2.9 Initial application of Maximum Sustainable Flow Rates	42
2.9.1 Planning	42
2.9.2 Design	42
<b>3 Definitions, methodologies and analysis</b>	<b>49</b>
3.1 Need for the Guide	49
3.2 Scope of the Guide	51
3.2.1 Motorways within large metropolitan cities	51
3.2.2 Maximum sustainable flow rates for metropolitan managed motorways	52
3.2.3 Caveat	53
3.3 Measurement methodology	54
3.3.1 Data	54
3.3.2 Methodology selection	54
3.3.3 Methodology application	57

3.4	Data and Site Observations	63
3.4.1	Data collection principles	63
3.4.2	Qualification of the data used	64
3.5	Measurement results	64
3.5.1	‘Capacity’	64
3.5.2	Maximum productivity	64
3.5.3	Probability of flow breakdown	65
3.5.4	Qualitative description of measurement sites	65
3.6	Comparison with International Design Values	81
3.7	Adjustment factors used for non-standard conditions	83
3.8	Evaluation of Managed Motorway Technology	84
3.9	Influence of tight curves	86
3.10	Future Investigation Work	88
	<b>Works Cited</b>	<b>89</b>

## List of Figures

Figure 1-1: General relationship between speed and flow rate (Source: HCM 2016).....	18
Figure 1-2: 'Highest observed traffic volume' versus 'Capacity value determined' (actual 1 hour speed and flow data measured over 1 month; Monash Freeway 4 lane cross-section 14587 IB, Jacksons Rd to Wellington Rd).....	20
Figure 1-3: Breakdown probability and productivity plotted against traffic flow .....	20
Figure 1-4: Breakdown probability, productivity and speed plotted against flow rate (2 lane cross-sections).....	23
Figure 1-5: Breakdown probability, productivity and speed plotted against flow rate (4 lane cross-sections).....	23
Figure 1-6: Breakdown probability and productivity plotted against per-lane flow rate for various cross-sections .....	24
Figure 2-1: Breakdown probability and productivity plotted against flow rate (2 lane cross-sections)	30
Figure 2-2: Breakdown probability and productivity plotted against flow rate (3 lane cross-sections)	31
Figure 2-3: Breakdown probability and productivity plotted against flow rate (4 lane cross-sections)	31
Figure 2-4: Breakdown probability and productivity plotted against flow rate (5 lane cross-sections)	31
Figure 2-5: Corridor Maximum Sustainable Flow Rates under various lane configurations.....	32
Figure 2-6: Breakdown probability and productivity plotted against flow rate (2 lane tunnels).....	34
Figure 2-7: Breakdown probability and productivity plotted against flow rate (3 lane tunnels).....	34
Figure 2-8: Breakdown probability and productivity plotted against flow rate (4 lane tunnels).....	35
Figure 2-9: Flow breakdown probability over 15 minutes for managed and unmanaged motorways (Warrigal Road (3 lanes, top) and Hallam Bypass (2 lanes, bottom), outbound direction) .....	37
Figure 2-10: Breakdown probability and productivity plotted against flow rate (2 lane cross-sections) .....	38
Figure 2-11: Breakdown probability and productivity plotted against flow rate (3 lane cross-sections) .....	38
Figure 2-12: Breakdown probability and productivity plotted against flow rate (4 lane cross-sections) .....	38
Figure 2-13: Breakdown probability and productivity plotted against flow rate (5 lane cross-sections) .....	39
Figure 2-14: Average vehicle speed for 2 lane cross-sections (100 km/h speed limit) .....	39
Figure 2-15: Average vehicle speed for 3 lane cross-sections (100 km/h speed limit) .....	40
Figure 2-16: Average vehicle speed for 4 lane cross-sections (100 km/h speed limit) .....	40
Figure 2-17: Average vehicle speed for 5 lane cross-sections (100 km/h speed limit) .....	40
Figure 2-18: Typical relationships between hourly volumes and AADT (Source: Austroads, 2013) ...	43

Figure 2-19: Typical relationships between hourly volumes and AADT (Source: HCM, 2016)..... 43

Figure 3-1: Vehicle trips along the M1 Corridor ..... 49

Figure 3-2: ‘Heat Plot’ from the Monash Freeway (inbound speed profile) ..... 58

Figure 3-3: Speed-density relation for cross-section 145871B (Monash Freeway - Jacksons to Wellington Road, inbound, 4 lanes) - measured values (1 hour rolling averages aggregated for density classes) and van Aerde curve approximation (elimination of downstream bottleneck influence – hence no very high densities) ..... 60

Figure 3-4: Speed-flow relation for cross-section 145871B (Monash Freeway - Jacksons to Wellington Road, inbound, 4 lanes) - measured values (1 hour rolling averages aggregated for density classes) and van Aerde curve approximation (elimination of downstream bottleneck influence) ..... 60

Figure 3-5: Carriageway productivity-density relation for cross-section 145871B (1 hour rolling averages aggregated for density classes) ..... 62

Figure 3-6: ‘Capacity’ and volume at maximum productivity for cross-section 145871B (1 hour rolling averages aggregated for density classes) ..... 63

Figure 3-7: Flow breakdown probability at Warrigal Road - before and after M1 Upgrade (7839 OB and 14547 OB, 15 min intervals) ..... 85

Figure 3-8: Flow breakdown probability on Hallam Bypass – before and after M1 Upgrade (14316 OB and 14456 OB, 15 min intervals) ..... 85

Figure 3-9: Example for a Productivity Frequency Plot (14316 OB and 14456 OB, 15 min intervals) 86

Figure 3-10: Car deceleration on curves (Austroads, 2016)..... 87

Figure 3-11: Speed reduction on curves (Harwood et al., 2014)..... 88

## List of Tables

Table 1-1: CM Levels of Service (LOS) – Values in brackets: Converted into metric units .....	25
Table 1-2: Static factors impacting on Maximum Sustainable Flow Rates .....	25
Table 1-3: Dynamic factors impacting on Maximum Sustainable Flow Rates .....	26
Table 2-1: Carriageway MSFR design values (veh/h) - managed motorways (gradient $s \leq 2\%$ ) .....	29
Table 2-2: Carriageway MSFR design values (veh/h) - managed motorways ( $2\% < \text{gradient } s \leq 3\%$ ) .....	30
Table 2-3: Carriageway MSFR design values (veh/h) - managed motorways ( $3\% < \text{gradient } s \leq 4\%$ ) .....	30
Table 2-4: Carriageway MSFR design values (veh/h) - managed motorways ( $4\% < \text{gradient } s \leq 5\%$ ) .....	30
Table 2-5: Tunnel MSFR design values (veh/h) – managed motorways (gradient $s \leq 2\%$ ).....	33
Table 2-6: Tunnel MSFR design values (veh/h) – managed motorways ( $2\% < \text{gradient } s \leq 3\%$ ) ....	33
Table 2-7: Tunnel MSFR design values (veh/h) – managed motorways ( $3\% < \text{gradient } s \leq 4\%$ ) ....	33
Table 2-8: Tunnel MSFR design values (veh/h) - managed motorways ( $4\% < \text{gradient } s \leq 5\%$ ) .....	33
Table 2-9: Carriageway MSFR design values (veh/h) – unmanaged motorways ( $s \leq 2\%$ ).....	35
Table 2-10: Carriageway MSFR design values (veh/h) - unmanaged motorways ( $2\% < \text{gradient } s \leq 3\%$ ).....	36
Table 2-11: Carriageway MSFR design values (veh/h) - unmanaged motorways ( $3\% < \text{gradient } s \leq 4\%$ ).....	36
Table 2-12: Carriageway MSFR design values (veh/h) - unmanaged motorways ( $4\% < \text{gradient } s \leq 5\%$ ).....	36
Table 2-13: van Aerde model parameters for different cross-sections .....	41
Table 3-1: ‘Capacity’ values .....	64
Table 3-2: ‘Capacity’ values compared to flows at maximum productivity .....	65
Table 3-3: Flow rates for different traffic flow breakdown probabilities (not normalised).....	65
Table 3-4: Example 1 - 2 lane cross-section (inside urban areas) .....	81
Table 3-5: Example 2 - 4 lane cross-section (inside urban areas) .....	81
Table 3-6: HGV-Percentage adjustment factors.....	83
Table 3-7: Gradient adjustment factors.....	83
Table 3-8: Before M1 Upgrade capacities .....	84



## Chapter 1: A new approach to assessing motorway section operation

**Chapter 1** explains the classical understanding of traffic flow and operational performance and establishes the need for a refined approach in light of customer expectations, road operator's targets and recent research results. It includes details on the reasoning behind the development of principles used for adoption of Maximum Sustainable Flow Rates to plan, design and manage motorways in real time to achieve optimum productivity.

**Chapter 2** provides guidance on the Maximum Sustainable Flow Rates for planning and design of motorways, and

**Chapter 3** contains details on the capacity, productivity and flow breakdown probability determination methodologies including the corresponding measurement results and other scientific considerations.





# 1 Assessing Motorway Section Operation

## 1.1 Customer expectations and road operator's targets

Market research and stakeholder engagement over many years have repeatedly confirmed that drivers and passengers expect the road network to deliver:

- Safe operating conditions
- Acceptable travel times (minimising delays - not necessarily delay free)
- Reliable travel times (consistent travel times for regularly travelled trips)

In summary, drivers and passengers want to have **'Good quality journeys'**.

In addition to servicing the individual desires of road users, a road operator needs to also deliver:

- Efficient utilisation of the road network, understanding it is a community asset with constraints in space and time and that the ability to build new roads is limited
- Safely and reliably servicing as many trips as possible per unit of space and time (optimisation of throughput and outflow)
- Minimisation of delays to enhance economic productivity and user utility across all road uses

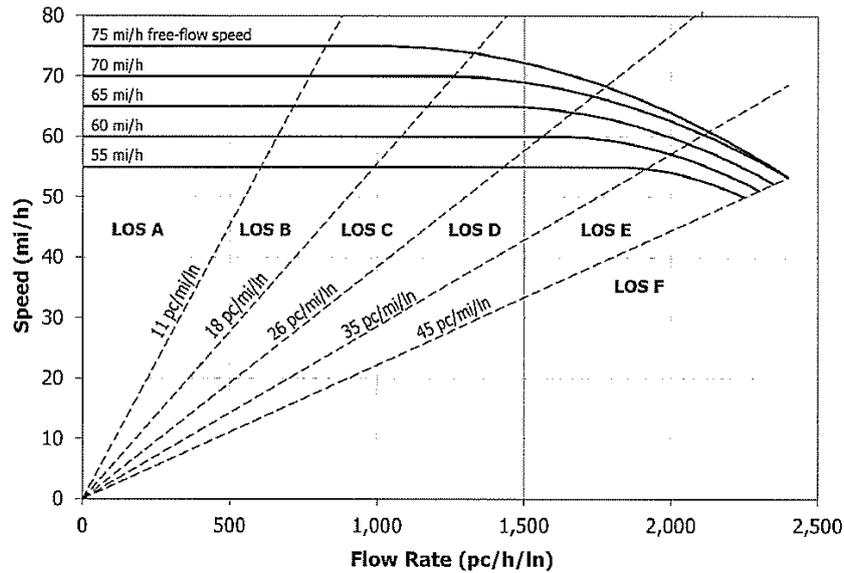
Taking into account these objectives, a common principle of road operators is therefore to manage their networks to provide **'Good quality journeys to many people'**.

Minimisation of flow breakdown and congestion should be a key objective in operating a road network since they impact on the quality of many journeys. Maximisation of an additional Key Performance Indicator (KPI) called 'Productivity' which is mathematically defined as the product of speed and flow should be another key objective since it supports the principle of providing 'Good quality journeys to many people'.

The role of 'Productivity' in network operation is recognised in the 'National Performance Indicators for Network Operations' framework (Austroads, 2007). It is a useful objective to maximize productivity because 'a high productivity is achieved if both speed and flow are maintained near maximum values, i.e. near free-flow speed and capacity flow'. Also, under current best practice vehicle operating regimes, efficient energy use aligns closely with the point where the motorway operates at maximum productivity.

## 1.2 Traffic flow and operational performance

The classical approach to understanding traffic flow on motorways focuses on three basic parameters used to describe the various traffic flow conditions: speed, flow rate (volume) and density. The most commonly used perspective is the relationship between speed and flow rate (Figure 1-1).



**Figure 1-1: General relationship between speed and flow rate (Source: HCM 2016)**

The USA Highway Capacity Manual 2016 (Transportation Research Board, USA Highway Capacity Manual, 2016), Exhibit 10-6 defines different density bands that in theory coincide with a certain traffic flow quality or Level of Service (LOS) provided to motorists (refer also to Section 1.8, Table 1-1). Drivers and passengers mainly notice whether they drive in uncongested or congested conditions.

The simplified illustrations in Figure 1-1 and the numbers in Table 1-1 are unclear about the increasing risk of a flow breakdown, i.e. about the risk of a transition from ‘free flow to ‘forced flow’ which usually occurs with an abrupt speed and flow reduction, from LOS D onwards. Flow breakdowns not only reduce speeds and flows, they also significantly increase the crash risk.

The USA Highway Capacity Manual 2016 on Page 4-21 defines ‘capacity’ of a system element as: *‘The maximum **sustainable** hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, environmental, traffic, and control conditions.’*

The definitions in other highway capacity manuals including in the German HCM (FGSV, 2015) are similar.

The capacity of a motorway system can be affected by various factors and can vary from section to section within a system. The HCM 2016 on Page 10-13 states that the notion of capacity on a freeway facility can be described as follows: *‘Freeway facility capacity is governed by the position and severity of active bottlenecks (i.e., segments with (demand to capacity ratio)  $v_d/c > 1.0$ ) along its length. Both characteristics vary over time and space, depending on the time-varying demand flow rates on each facility segment.’*

There will be cross-sections within a freeway facility where higher hourly flow rates (even if averaged over statistically relevant periods) than at bottlenecks have been observed which consequently also exceed the Maximum Sustainable Flow Rates (MSFR) listed in this Guide. However, considering the above HCM statement, this is irrelevant for the planning and design task for which these values were developed and will be used.

According to the HCM 2016, capacity is the flow rate at the end point of the speed flow relationships as shown in Figure 1-1. This flow rate coincides with the occurrence of a considerable probability of flow breakdown over a short period (e.g. 15 minutes) and even more so over an extended peak period (e.g. 3 hours). Therefore, there is a **considerable inconsistency between capacity values**

**as stated in the HCM and the above definition** which requires that capacity is a **sustainable** flow rate.

The most meaningful way to illustrate the 'sustainability' (or stability) of traffic at a certain flow rate is to characterise it by the probability of a flow breakdown that coincides with that flow rate. A new approach that has been foreshadowed in the new HCM 2016 is the determination of the maximum sustainable flow rate based on the probability of flow breakdown.

## 1.3 Data and observations

### 1.3.1 International perspective

The traditional approach to determining 'capacity' and Level of Service (LOS) is to fit a curve to a point cloud representing individual speed-flow measurements. Capacity is then defined as the highest flow rate of that curve and the different LOS are density ranges within it. However, this approach is now changing internationally with the availability of better quality data.

Recognition that 'capacity varies over time' or that 'capacity is stochastic' can be seen as state-of-the-art. Elefteriadou and Lertworawanich (Elefteriadou and Lertworawanich, 2003) recommend 'that the breakdown flow be used' for the definition of capacity, in particular because 'it is consistent with the current implication that capacity is the boundary between non-congested and congested conditions'. Analysis of the pre-breakdown flow is now the recommended HCM approach for capacity estimation (refer to HCM 2016, Page 26-18). The pre-breakdown flow rate does not have a fixed value since breakdowns are stochastic in nature and could occur following a range of flow rates. The capacity values stated in the Dutch HCM (Henkens and Heikoop, 2015) are also based on the analysis of pre-breakdown flow rates and their distribution.

### 1.3.2 VicRoads perspective

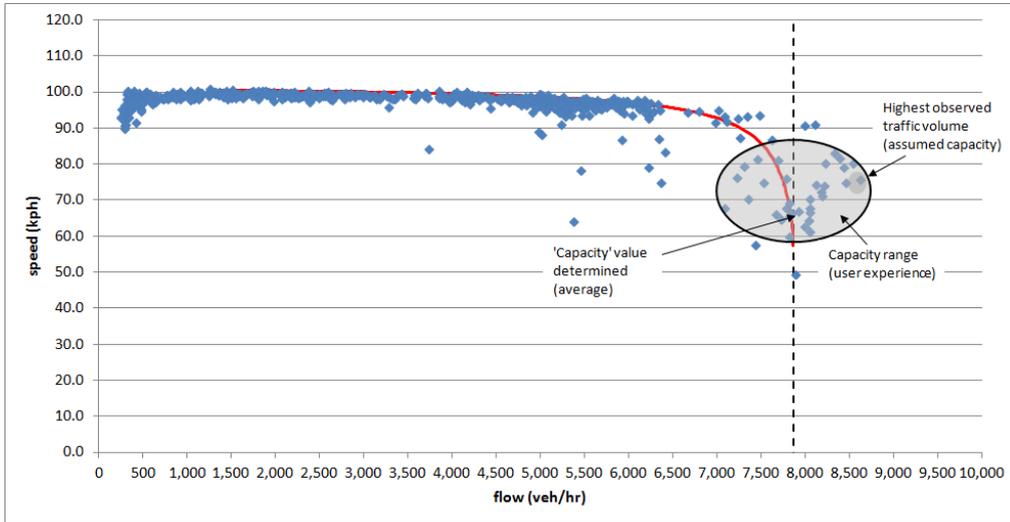
VicRoads has analysed extensive data available on its motorways and determined 'capacity' values for managed motorway cross-sections. This was based on Monash and West Gate Freeway traffic data (volume, speed and occupancy). Capacity was first determined as the maximum of the curve that was fitted to a point cloud representing individual speed-flow measurements. This leads to a better reflection of a range of dynamic factors that impact on traffic flow and hence 'capacity', than using randomly selected high traffic volumes as a representative motorway 'capacity'.

The problem of randomly selected high traffic volumes ('assumed capacity') versus an appropriate representation of different traffic states in the relevant area ('average') is illustrated in Figure 1-2. It needs to be understood that the speed-flow combinations in the 'Capacity range' are widely spread. They need to be interpreted and represented in a meaningful way.

Capacity is different for every site that is frequently oversaturated, although there is the perception that it is constant for a given number of lanes. Also, when analysing detailed traffic data from a motorway site, it is starkly evident that no two days ever have the same maximum throughput/flow rate. Plotting the fundamental speed-flow and density-flow relationships on a daily basis clearly shows different shapes occur every day. Further, when breaking a day into shorter time periods, it is clear that different relationships occur at different times of a day.

The definition and methodologies used to determine capacity as well as typical design values clearly indicate that these values are an outcome of some form of historical trend, and are therefore an abstraction from day to day experience. The methodologies that are based on an extended data set ( $\approx$  1 month) mask, or smooth out, the fine grain variations that are inherent in motorway operations. It therefore needs to be asked: 'What bearing does capacity have on the day to day operations and on design and control decisions?' In the context of this question, capacity as well as typical design values usually comprise a range of values so should be seen as averages or as an indication of a motorway system's performance capability.

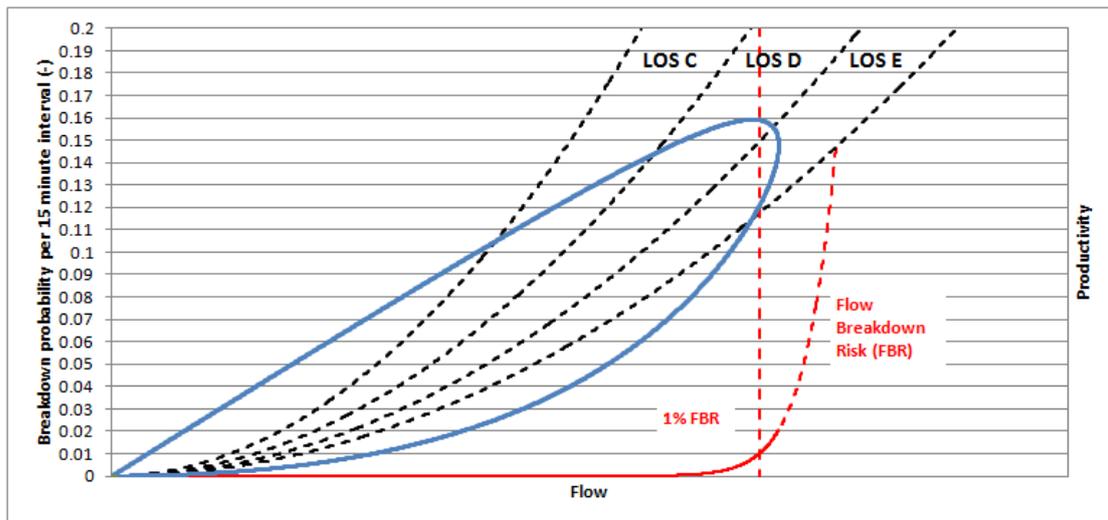
From acknowledging the dynamic if not random nature of traffic flow and capacity, it follows that a stochastic approach provides valuable insights into the expected outcomes for drivers and passengers as well as for road operators.



**Figure 1-2: ‘Highest observed traffic volume’ versus ‘Capacity value determined’ (actual 1 hour speed and flow data measured over 1 month; Monash Freeway 4 lane cross-section 14587 IB, Jacksons Rd to Wellington Rd)**

With the aim of defining Maximum Sustainable Flow Rates that better reflect the HCM definition, additional analysis undertaken by VicRoads has therefore also involved understanding the probability of flow breakdown occurring at different flow levels. Generally, it is observed that stable and relatively high traffic flows are achieved before the probability of flow breakdown starts to significantly increase.

VicRoads has also explored the relationship between capacity and ‘Productivity’ (refer to Section 1). Figure 1-3: shows the relationship between flow rate and traffic flow breakdown probability as well as productivity.



**Figure 1-3: Breakdown probability and productivity plotted against traffic flow**

When studying the different KPI’s and their relationship, it has become evident that:

- The flow breakdown risk starts to exponentially increase from a flow rate close to that at maximum productivity

- Maximum productivity occurs at a flow rate lower than at 'capacity'
- High speeds can be sustained before maximum productivity (with the corresponding flow-on effects regarding the minimisation of travel times and delays)
- High flows can be achieved from a density or flow rate close to that at maximum productivity

The requirements of both the road user and the operator are most aligned when the flow is close to the point of maximum productivity. The next section develops an approach that identifies this point.

## 1.4 Maximum Sustainable Flow Rate

As a conclusion of the above considerations, absolute 'capacity' values as included in most international guidelines are not considered to be the most suitable indicator to meet customer expectations or to achieve road operator's targets as they fail to coincide with the needed sustainability of flow rates.

A more appropriate KPI is the concept of Maximum Sustainable Flow Rates (MSFR). A road network operator's objective is to deliver 'Good quality journeys to many people'. This statement implies that achieving reliability and productivity are important aspects of managing a motorway. A recommendation regarding the use of Maximum Sustainable Flow Rates therefore needs to consider a relatively low probability of a flow breakdown and achieving high productivity.

When good quality motorway data is analysed, the point where the probability of a flow breakdown starts to significantly (or in fact exponentially) increase is relatively clearly defined. Once this point has been exceeded, the flow breakdown risk and hence the expected traffic flow quality will inevitably deteriorate. In contrast, productivity can be relatively high on either side of its maximum.

**Chapter 2** provides guidance on the Maximum Sustainable Flow Rates for planning, design and operation of motorways. The recommended flow rates result from extensive analysis of actual motorway performance data and are based on

1. 1% breakdown probability per 15 minute interval (or around 10% breakdown probability for a 3 hour peak period), and
2. Maximisation of productivity.

They also consider the relationship of both KPI's to the classically defined capacity.

The adopted approach takes into account that as a road manager, VicRoads needs to be accountable for network performance (i.e. speed and flow) achieved over periods of high demand. If a peak period extends over 3 hours, it is important that there is understanding of what motorists actually experience across such a period. Reaching high speeds and flows in the first hour and observing significantly reduced speeds and flows (with unsafe stop-start conditions) for the remaining 2 hours does not meet customer expectations or road operator objectives.

It is important that Maximum Sustainable Flow Rates are assessed on an ongoing basis, as Melbourne's Managed Motorway network as well as vehicle technology evolve and mature to ensure that any improvements in operational performance can be incorporated into practitioner guidance documents.

## 1.5 Factors affecting Maximum Sustainable Flow Rates

It is important that the analysis methods used effectively take into account the whole range of factors that influence 'capacity' and therefore also productivity and Maximum Sustainable Flow Rates.

Table 1-2 in Section 1.8 indicates the (quasi-) static conditions that are commonly seen as influencing traffic flow and are hence considered for planning and design when analysing existing or planned motorway segments. Table 1-3 in the same section indicates that there is also a wide range of dynamic factors impacting on Maximum Sustainable Flow Rates.

## 1.6 Detailed description of Key Performance Indicators and their relationship

Combining the 'Probability of flow breakdown' and 'Maximisation of productivity' objectives informs appropriate control ranges in order to ensure motorway operation remains stable and productive and informs decision making required for effective planning and design of urban motorways. Moreover, initial investigations indicate that there is a cluster of crashes on urban motorways that occurs when conditions start to transition from stable to unstable flows (i.e. before the probability of flow breakdown starts to significantly increase).

Figure 1-4 and Figure 1-5 are typical diagrams showing a managed motorway's flow breakdown risk, productivity and speed as a function of the flow rate for 2 lane and 4 lane cross-sections (gradient  $\leq$  2%, HGV-percentage = 15%). They also show the relationship with the traditional HCM LOS density bands, as defined in Table 1-1 ('Urban').

The following key points provide some detailed information on the traffic state at selected flow rates as shown in the diagrams which are based on the corresponding HCM descriptions (refer to Table 1-1 in Section 1.8):

- Point 1: Traffic speeds at or near the free-flow speed with noticeably restricted freedom to manoeuvre within the traffic stream. Relatively low flow rates. No breakdown risk.
- Point 2: Declining speeds due to increasing flows with seriously limited freedom to manoeuvre within the traffic stream. Relatively high flow rates. Very low breakdown risk.
- Point 3: Productivity which is the combined value of (declining) speeds and (increasing) flow rates close to its maximum. Probability of flow breakdown starting to significantly increase.
- Point 4: Operations at 'capacity' which are highly volatile because there are virtually no usable gaps within the traffic stream; unstable with no ability to dissipate even the most minor disruption. High probability of flow breakdown which is increasing exponentially.

From the analysis carried out on Melbourne's motorway network, it is considered that the adoption of 1% breakdown probability per 15 minute interval (or around 10% for a 3 hour peak period) is reasonable to ensure good network reliability and productivity that is close to its maximum (Point 3). This method correlates fairly well with methods that adopt a flow rate equivalent to approximately 90% of traditional capacity or maximum flow rate.

As can be seen from Figure 1-4 and Figure 1-5, the probability of flow breakdown is starting to significantly increase beyond Points 2 and 3 within LOS D. Only effective active control as provided by managed motorway technology can keep operations at these points. Without control, increased demand will very quickly push the traffic state beyond Point 4. Any motorway with an expected Level of Service (LOS) beyond LOS C should operate as a managed facility to ensure Maximum Sustainable Flow Rates can be achieved if maximisation of productivity is an objective. Implementing managed operations needs to be considered in a corridor/network context and not just applied in isolation (refer also to (Gaffney et al., 2015)).

Figure 1-6 on a per-lane basis combines the corresponding curves for 2- to 5-lane cross-sections into one graph. It illustrates the systematically decreasing Maximum Sustainable Flow Rates (MSFR) with increasing number of carriageway lanes which is particularly due to the exponential growth in lane changing activity needed to fill all the lanes including the inner ones to 'capacity'. Chapter 2 provides detail on the respective MSFR values. Lane changing is one of the dynamic factors affecting MSFR as listed in Table 1-3 in Section 1.8.

Part 3: VicRoads Motorway Capacity Guide

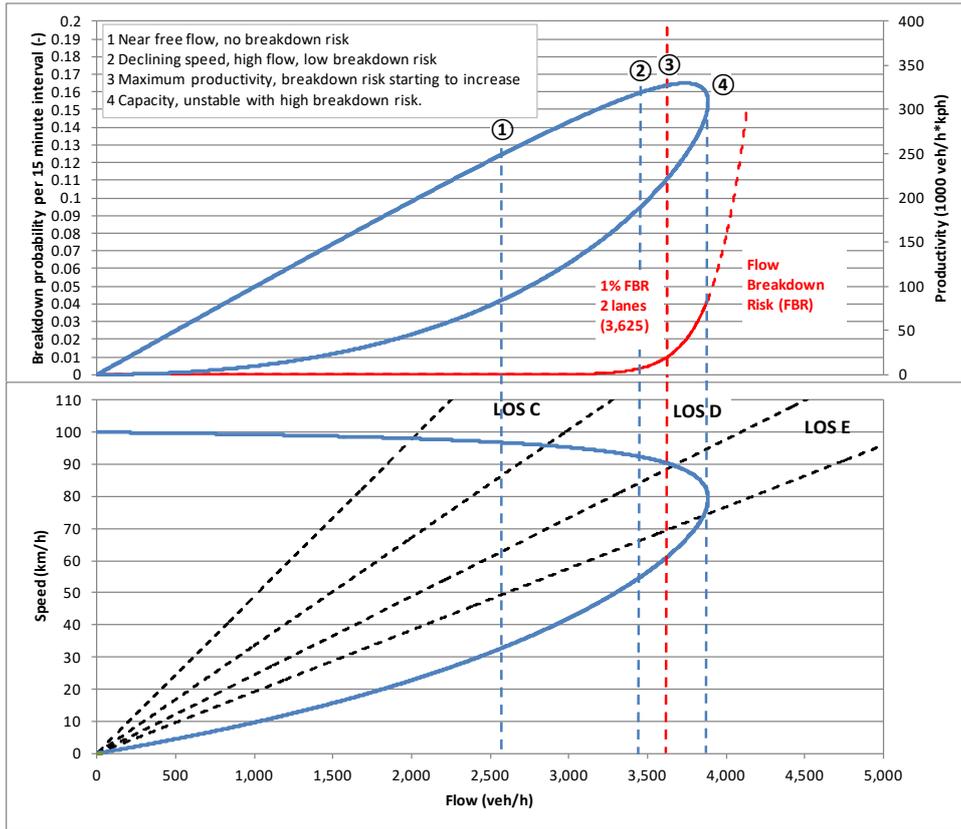


Figure 1-4: Breakdown probability, productivity and speed plotted against flow rate (2 lane cross-sections)

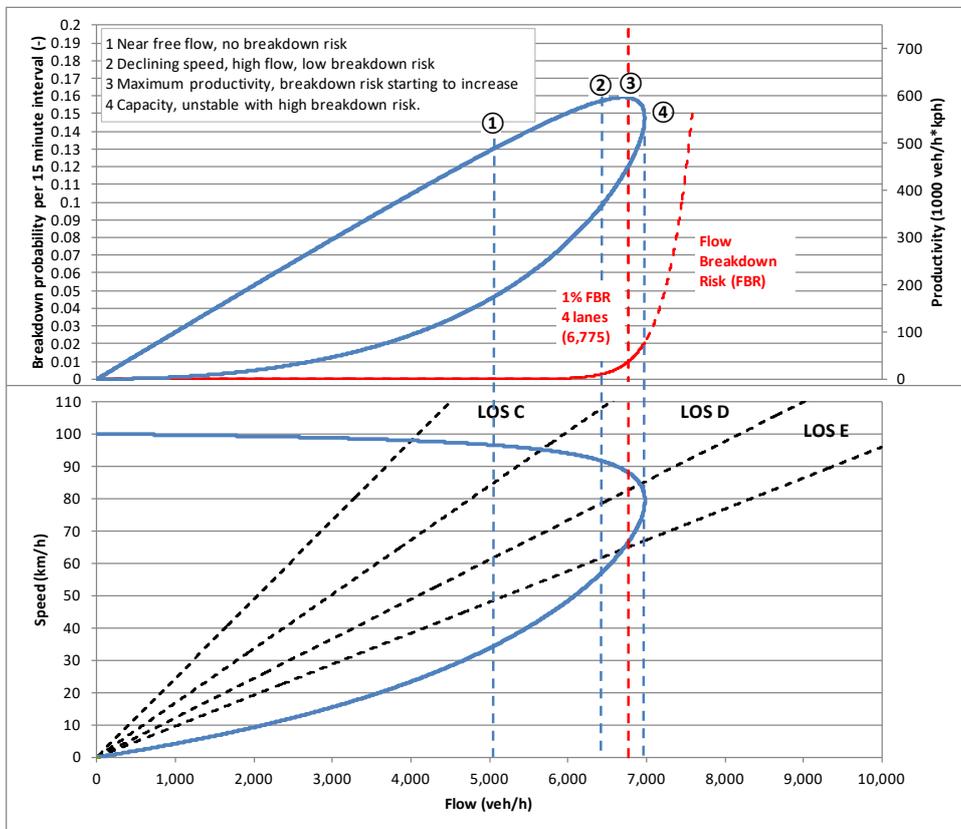
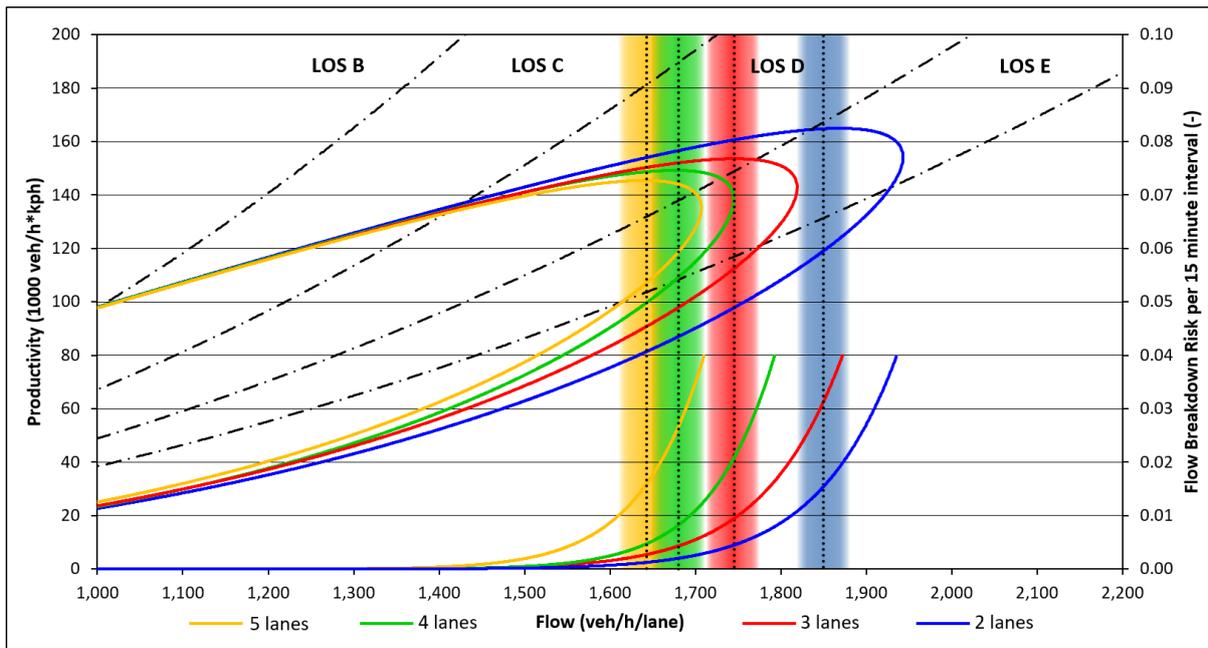


Figure 1-5: Breakdown probability, productivity and speed plotted against flow rate (4 lane cross-sections)



Note: Dotted lines show the points of maximum productivity and coloured bands illustrate reasonable operational ranges

**Figure 1-6: Breakdown probability and productivity plotted against per-lane flow rate for various cross-sections**

## 1.7 Concluding remarks

Absolute ‘capacity’ values as included in most international guidelines are no longer considered to be a suitable indicator to meet customer expectations or to achieve road operator’s targets as they fail to coincide with the needed sustainability of flow rates. Adjustments of such ‘capacity’ values (e.g. use 90% of it for ‘design’) may coincide with sustainable conditions but the selection of the corresponding factors is not transparent. There is a need to determine Maximum Sustainable Flow Rates (MSFR) which are based on objective criteria. While it is only slowly finding its way into Highway Capacity Manuals around the world, the concept of linking MSFR to the probability of flow breakdown is a transparent approach since flow breakdowns conflict with customer expectations and road operator’s targets, both in terms of safety and efficiency as well as environmental outcomes. VicRoads has adopted this concept in conjunction with the HCM for motorway planning and design tasks.

Any motorway with an expected peak hour traffic flow quality beyond LOS C should be planned and designed to operate as a managed facility since it is necessary to control traffic in the area where the probability of flow breakdown is starting to significantly increase.

Besides use for planning, design and operation, the methodologies presented in this Guide should form the basis for the evaluation of investment in managed motorway technology which can be major (upgrade to a managed motorway) or minor (re-tuning of the City Wide Coordinated Ramp Metering (CWCRM) or other algorithms and parameters in scope). Details on a potential approach to evaluation methodologies can be found in Section 3.8.

## 1.8 LOS density bands and factors influencing traffic flow

Table 1-1 shows Level of Service (LOS) density bands for urban and rural conditions as included in the USA Highway Capacity Manual 2016 (Transportation Research Board, USA Highway Capacity Manual, 2016).

LOS	Freeway Facility Density (pc/mi/ln)		
		Urban	Rural
A	Free-flow operations where vehicles are almost completely unimpeded in their ability to manoeuvre within the traffic stream.	<= 11 (7)	<= 6 (4)
B	Reasonably free flow conditions where the ability to manoeuvre within the traffic stream is only slightly restricted.	11-18 (7-11)	6-14 (4-9)
C	Traffic speeds at or near the free-flow speed with noticeably restricted freedom to manoeuvre within the traffic stream.	18-26 (11-16)	14-22 (9-14)
D	Declining speeds due to increasing flows with seriously limited freedom to manoeuvre within the traffic stream.	26-35 (16-22)	22-29 (14-18)
E	Operations at or near capacity which are highly volatile because there are virtually no usable gaps within the traffic stream; unstable with no ability to dissipate even the most minor disruption.	35-45 (22-28)	29-39 (18-24)
F	Unstable flow with queues forming behind bottlenecks. Breakdowns occur for a number of reasons.	> 45 (28)	> 39 (24)

**Table 1-1: CM Levels of Service (LOS) – Values in brackets: Converted into metric units**

Table 1-2 indicates (quasi-) static conditions and Table 1-3 the wide range of dynamic factors that influence traffic flow and Maximum Sustainable Flow Rates.

<b>Cross section</b>
<b>Number of lanes</b>
Lane and Shoulder Widths: Particularly lane widths narrower than 3.25m (or 3.35m), also dependent on the length of the section with narrow lanes/shoulders
<b>Visibility conditions: E.g. drivers field of view constrained by high noise walls or open with clear view</b>
<b>Alignment</b>
Gradient: Particularly >2% and sustained for >750m (number indicative)
Curvature: Particularly radius <700m (number indicative)
Sags and crests: Particularly sag curves in tunnels
<b>Traffic</b>
Long-term average share of different vehicle types (in particular Heavy Goods Vehicle (HGV) percentage)
<b>Location</b>
Impacting on the share of commuters that are familiar with the road (high share increases capacity)
<b>Other physical attributes</b>
Merge/diverge tapers, acceleration/deceleration lengths, auxiliary lanes, braiding, collector-distributor lanes etc

**Table 1-2: Static factors impacting on Maximum Sustainable Flow Rates**

<b>Operational practice</b>
Traffic management interventions: Dynamic Traveller Information including VMS messaging (directly affects route choice, trip length and hence Origin-Destination (OD) patterns with the corresponding impacts on lane changing), lowering speed limits, overtaking bans, closing lanes or ramps
Ramp metering actions: Independent and coordinated actions can impact on the use of available capacity, both positively and negatively (i.e. uncontrolled vehicle release at on-ramps when queues are excessive can cause flow breakdown on the mainline and ultimately a loss in productivity across the entire road network)
Maintenance: Routine maintenance activities, road works and presence of work zones (e.g. slow moving maintenance vehicles or fewer and narrow lanes)
Enforcement regime (e.g. harmonised speeds increase capacity)
<b>Driver Behaviour</b>
Increased headways: Environmental conditions including reduced visibility caused by rain or poor surface roughness
Heavy braking, large variation in vehicle speeds, or harsh lane changing manoeuvres: Can cause a perturbation causing flow breakdown
Rubbernecking: Can cause a perturbation causing flow breakdown (e.g. caused by speed cameras)
Compliance with Road Rules: Speed limit, overtaking, lane use and ramp metering compliance
<b>Vehicles</b>
Short-term (e.g. 1 min) random variations of different variables (e.g. spikes in traffic demand, changes in local origin-destination relations, heavy vehicle cluster, increased lane changing intensity) can cause a perturbation causing flow breakdown
Broken down vehicles
Objects on the road (e.g. vehicle parts)

**Table 1-3: Dynamic factors impacting on Maximum Sustainable Flow Rates**

## Chapter 2: Application of Maximum Sustainable Flow Rates

**Chapter 2** provides guidance on the Maximum Sustainable Flow Rates for planning and design of motorways with a gradient of up to 5% and a Heavy Goods Vehicle (HGV) share of up to 30% (speed limit of 100 km/h or 80 km/h). Different conditions including short lanes and steeper gradients or higher HGV percentages require careful consideration of all significant influencing factors.

**Chapter 1** includes details on the reasoning behind the adoption of Maximum Sustainable Flow Rates to plan, design and manage motorways, and

**Chapter 3** contains details on the capacity, productivity and flow breakdown probability determination methodologies including on the corresponding measurement results and other scientific considerations.





## 2 Maximum Sustainable Flow Rates

### 2.1 Why are Maximum Sustainable Flow Rates needed?

Maximum Sustainable Flow Rates (MSFRR) relate to the needed reliability and productivity that are important to customers and road operators. MSFR included in the following sections are intended to meet the following objectives:

- 1% breakdown probability per 15 minute interval results in a breakdown probability per 3 hour peak period of around 10%. This is seen as an acceptable risk (noting that this percentage equates to flow breakdown approximately once per fortnight);
- Productivity is close to its maximum at the corresponding flow rate; and
- This method correlates fairly well with methods that adopt a flow rate equivalent to approximately 90% of capacity or maximum flow rate.

Planning and design based on MSFR provides the best service to a large number of drivers and passengers. Use of MSFR enables the road operator to deliver a safe and reliable road system and maximise productivity on the road network. As explained in Section 1.5, traffic flow indicators such as capacity, probability of flow breakdown and productivity are impacted by a range of static and dynamic factors that cause spatial (i.e. site specific) and temporal fluctuations. The stated MSFR (i.e. the flow rates at 1% probability of flow breakdown) are typical and each site will be slightly different. The adopted flow rates coincide with on average 92% of the traditional capacity (refer to Table 3-3).

### 2.2 Maximum Sustainable Flow Rates for managed motorways

#### 2.2.1 Carriageways

The Maximum Sustainable Flow Rates (MSFR) included in Table 2-1 to Table 2-4 should be used for the design of surface managed motorway sections. These rates are based on measured flow rates at 1% breakdown probability. Details on the MSFR determination methodology can be found in Sections 3.3.2 'Methodology selection' ('Capacity (Approach 2, Probability of flow breakdown)') and 3.3.3 'Methodology application'.

Figure 2-1 to Figure 2-4 show a managed motorway's Flow Breakdown Risk (FBR) and productivity as a function of the flow rate, for 2 lane to 5 lane cross-sections for standard conditions (i.e. gradient  $\leq 2\%$  and HGV-percentage = 15%). FBR curves for other gradients or HGV-percentages can be produced by changing the flow rate (value in the x-axis) proportionally to the change in Maximum Sustainable Flow Rates as included in Figure 2-1 to Figure 2-4.

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	4,175	3,975	3,800	3,625	3,475	3,325	3,200
3	6,050	5,775	5,525	5,250	5,050	4,850	4,625
4	7,800	7,450	7,125	6,775	6,500	6,225	5,975
5	9,275	8,875	8,475	8,050	7,750	7,425	7,100

**Table 2-1: Carriageway MSFR design values (veh/h) - managed motorways (gradient  $s \leq 2\%$ )**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3,950	3,775	3,625	3,450	3,300	3,175	3,025
3	5,750	5,500	5,250	5,000	4,800	4,600	4,400
4	7,400	7,075	6,750	6,425	6,175	5,925	5,675
5	8,800	8,425	8,050	7,650	7,350	7,050	6,750

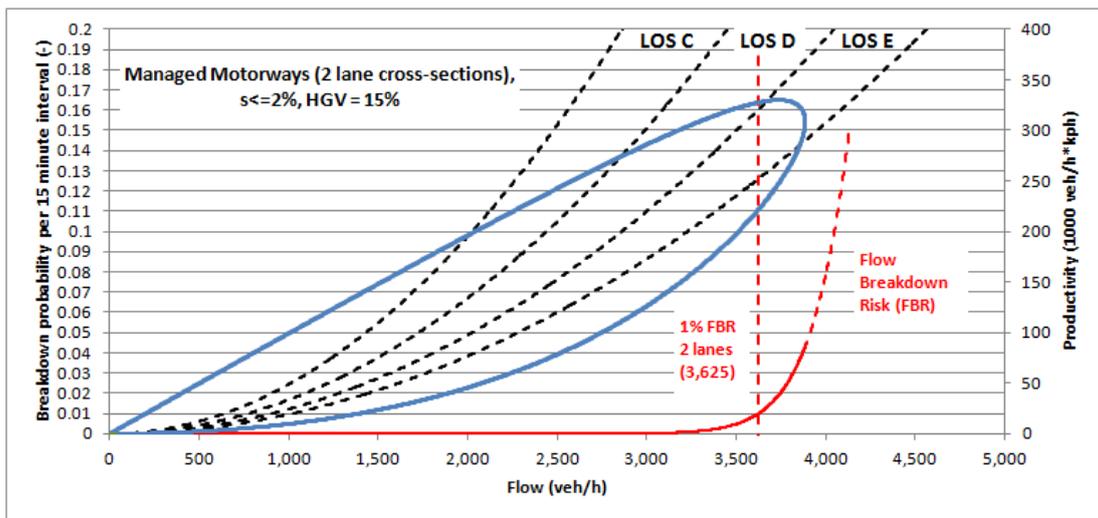
**Table 2-2: Carriageway MSFR design values (veh/h) - managed motorways (2% < gradient s <= 3%)**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3,750	3,600	3,425	3,250	3,125	3,000	2,875
3	5,450	5,200	4,975	4,725	4,550	4,350	4,175
4	7,025	6,700	6,400	6,100	5,850	5,600	5,375
5	8,350	7,975	7,625	7,250	6,975	6,675	6,375

**Table 2-3: Carriageway MSFR design values (veh/h) - managed motorways (3% < gradient s <= 4%)**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3,450	3,300	3,150	3,000	2,900	2,775	2,650
3	5,025	4,800	4,575	4,375	4,200	4,025	3,850
4	6,475	6,175	5,900	5,625	5,400	5,175	4,950
5	7,700	7,350	7,025	6,700	6,425	6,150	5,875

**Table 2-4: Carriageway MSFR design values (veh/h) - managed motorways (4% < gradient s <= 5%)**



**Figure 2-1: Breakdown probability and productivity plotted against flow rate (2 lane cross-sections)**

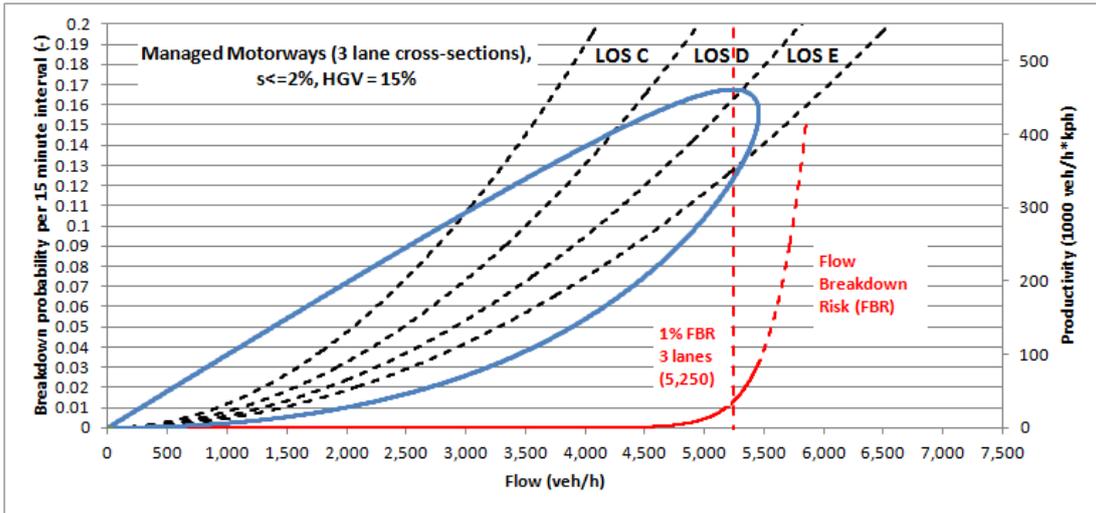


Figure 2-2: Breakdown probability and productivity plotted against flow rate (3 lane cross-sections)

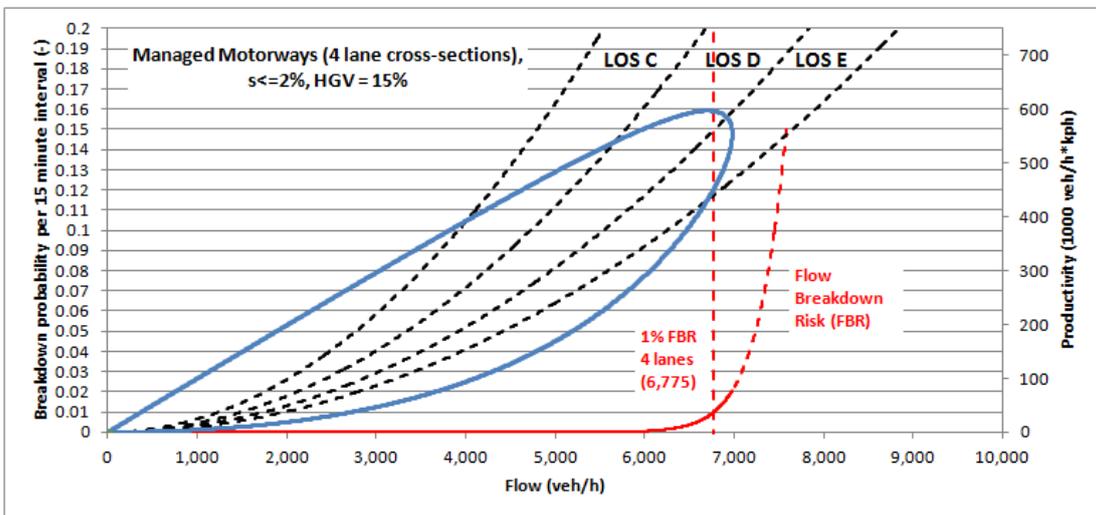


Figure 2-3: Breakdown probability and productivity plotted against flow rate (4 lane cross-sections)

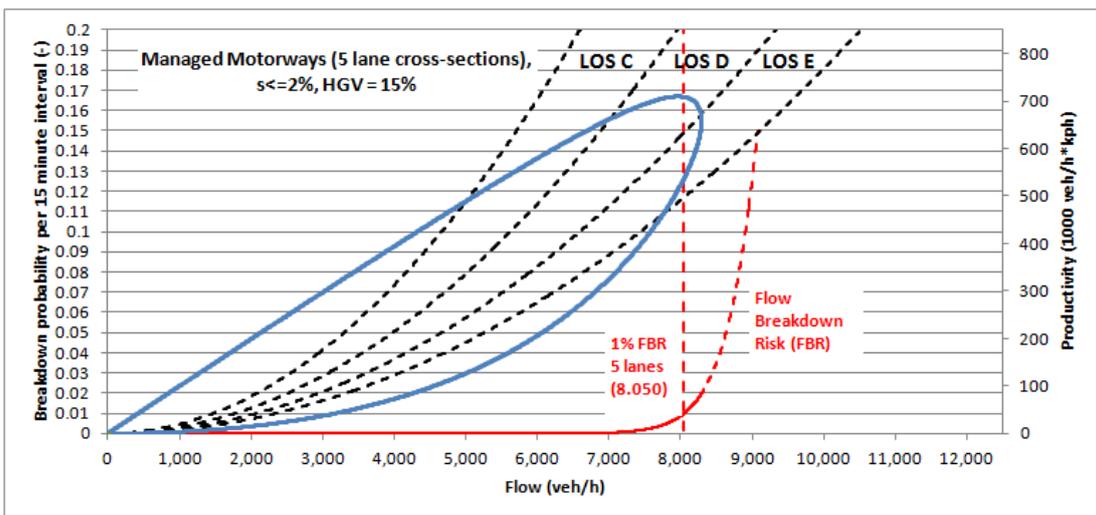


Figure 2-4: Breakdown probability and productivity plotted against flow rate (5 lane cross-sections)

## 2.3 Corridors

Figure 2-5 shows indicative corridor Maximum Sustainable Flow Rates under various lane configurations for standard conditions (i.e. gradient  $\leq 2\%$  and HGV-percentage = 15%). For example, it illustrates that two 2 lane carriageways (Maximum Sustainable Flow Rate = 7,250 veh/h) are more efficient than one 4 lane carriageway (Maximum Sustainable Flow Rate = 6,775 veh/h). The extrapolated trend (for combinations including 6 and 7 lanes) is a reduction of per lane Maximum Sustainable Flow Rate of 4% with every additional lane. This is consistent with measurements on Melbourne’s Managed Motorway network and international research and takes into account the increasing number of lane changes required to fill all lanes to capacity as more lanes are added.

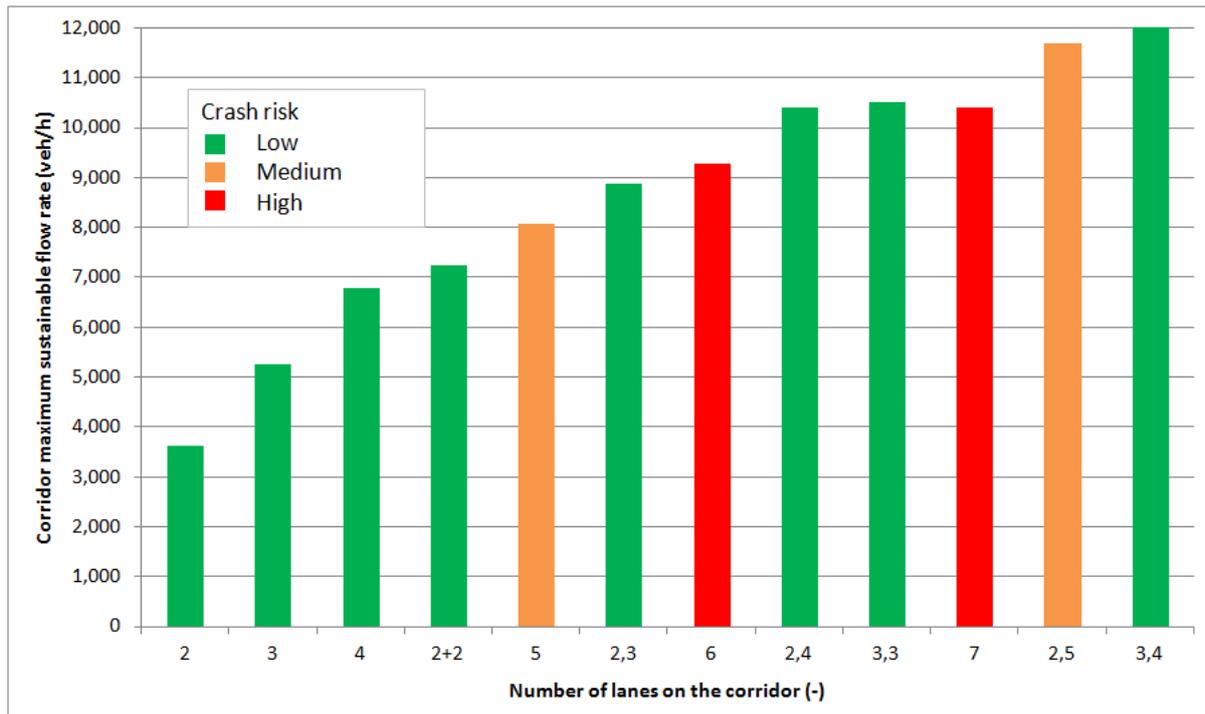


Figure 2-5: Corridor Maximum Sustainable Flow Rates under various lane configurations<sup>1</sup>

## 2.4 Tunnel sections

Capacity analysis for tunnels is of particular importance since they are often a suitable means for the closure of gaps in the metropolitan motorway network and also to respond to difficult terrain and sensitive community and environmental issues. Strong sites of frequent oversaturation can form in tunnels, in particular at tunnel entrances (portals) and at sags and grades which are often difficult to interpret by motorists in a tunnel environment because of different environmental conditions when compared with surface motorways (e.g. missing horizon, increased noise level limiting the ability to hear feedback from vehicles).

The MSFR included in Table 2-5 to Table 2-8 should be used for managed motorway sections in tunnels. These rates are based on measured flow rates at 1% breakdown probability per 15 minute interval for the reasons outlined in Sections 2.1 and 2.2.1. There are arguments for and against adopting a higher tolerable breakdown probability for tunnels which may neutralise each other: On one hand, tunnel infrastructure is much more expensive than surface motorway sections. On the other hand, flow breakdown in tunnels should be avoided because it coincides with a higher safety risk.

<sup>1</sup> International literature (Kononov et al., 2008) suggests that accident rates grow with increasing carriageway lane number; this is in particular due to the exponentially increasing number of lane changes needed to fill all lanes

Measurements were only undertaken in 3 lane tunnels since no data from other managed motorway tunnels was available. All values and curves stated below were proportionally factored up or down for 2 lane and 4 lane tunnels. The rationale for this is that driver behaviour (in particular lane changing activity) is different to surface motorway sections. There is currently no evidence for a change in per lane 'capacity' or Maximum Sustainable Flow Rates (MSFR) with increasing or decreasing carriageway lane number.

In order to achieve optimum traffic flow outcomes the following conditions should be met (refer also to (Broeren et al., 2010)):

Tunnel entrance:

- Merging completed 200 metres before the entrance portal (avoid intense lane changing activity at the tunnel entrance portal)
- Minimisation of signage at the entrance portal (avoid distraction and sudden braking manoeuvres when becoming aware of the tunnel entrance which is often perceived as a 'black hole' )

Tunnel exit:

- Diverging starting not earlier than 200 metres from the exit portal

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3800	3650	3475	3325	3175	3050	2925
3	5725	5475	5225	4975	4775	4575	4375
4	7625	7300	6950	6625	6350	6100	5825

**Table 2-5: Tunnel MSFR design values (veh/h) – managed motorways (gradient s <= 2%)**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3625	3450	3300	3150	3025	2900	2775
3	5425	5200	4950	4725	4525	4350	4150
4	7250	6925	6600	6300	6050	5800	5550

**Table 2-6: Tunnel MSFR design values (veh/h) – managed motorways (2% < gradient s <= 3%)**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3425	3275	3125	2975	2850	2750	2625
3	5150	4925	4700	4475	4300	4125	3925
4	6850	6550	6275	5975	5725	5475	5250

**Table 2-7: Tunnel MSFR design values (veh/h) – managed motorways (3% < gradient s <= 4%)**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3150	3025	2875	2750	2650	2525	2425
3	4750	4550	4325	4125	3950	3800	3625
4	6325	6050	5775	5500	5275	5050	4850

**Table 2-8: Tunnel MSFR design values (veh/h) - managed motorways (4% < gradient s <= 5%)**

Figure 2-6 to Figure 2-8 show a tunnel's flow breakdown risk (FBR) and productivity as a function of the flow rate, for 2- to 4-lane cross-sections.

The diagrams shown here are for gradients of 4% to 5% since this is considered to be the most likely case under Australian metropolitan conditions (e.g. tunnels for river crossings or underpasses of sensitive environmental areas). FBR curves for other gradients (e.g.  $s \leq 2\%$ ) or HGV-percentages can be produced by changing the flow rate (value in the x-axis) proportionally to the change in Maximum Sustainable Flow Rates as included in Table 2-5 to Table 2-8 (e.g. multiply the flow rates for the different flow breakdown risks by a Factor  $x = 4,975/4,125$ ).

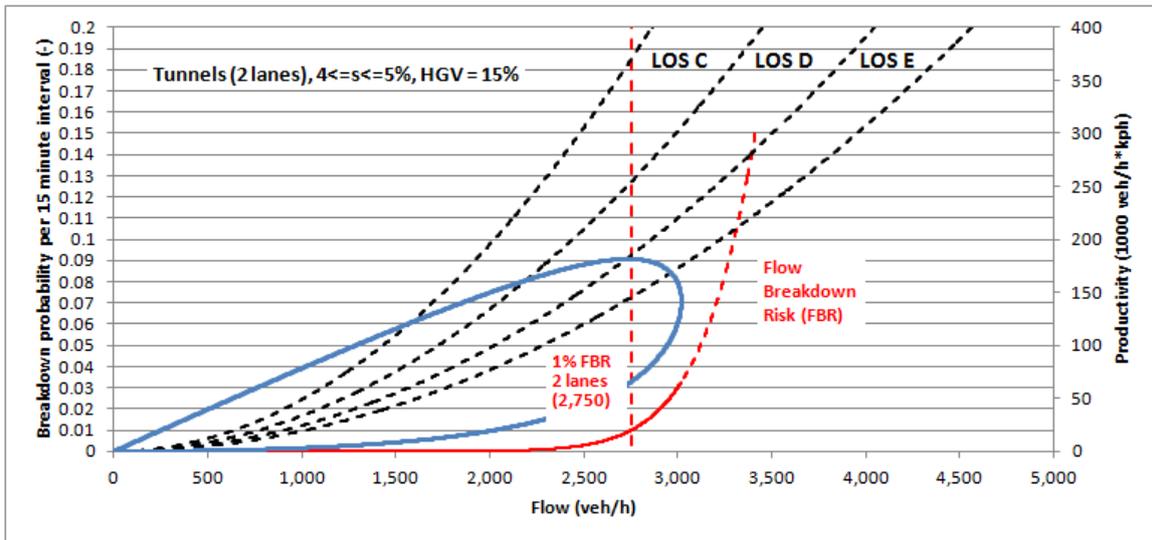


Figure 2-6: Breakdown probability and productivity plotted against flow rate (2 lane tunnels)

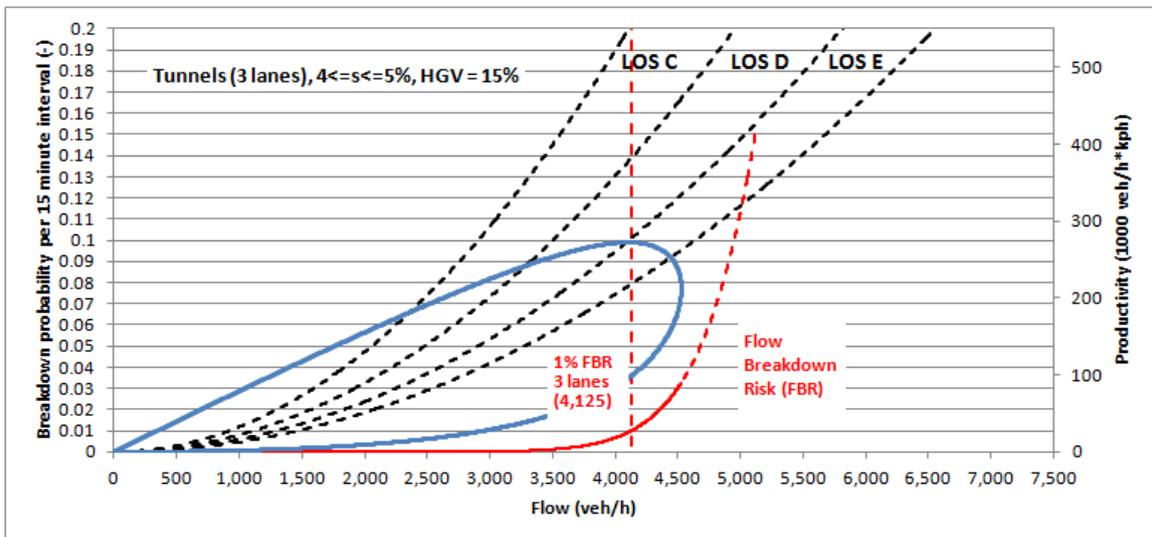


Figure 2-7: Breakdown probability and productivity plotted against flow rate (3 lane tunnels)

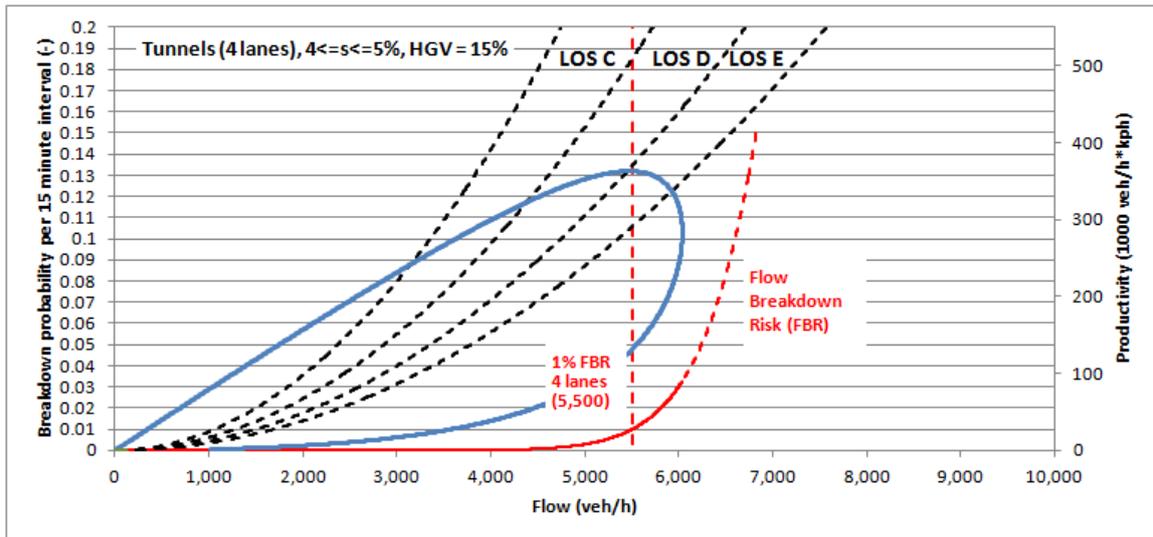


Figure 2-8: Breakdown probability and productivity plotted against flow rate (4 lane tunnels)

## 2.5 Maximum Sustainable Flow Rates for unmanaged motorways

Based on a comparison of Pre- and Post M1 Motorway Upgrade per lane capacities, the Maximum Sustainable Flow Rates included in Section 2.2.1 should be reduced by 15% for the design of unmanaged motorway surface sections (refer to Table 2-9 to Table 2-12). The managed motorways values and curves were correspondingly factored down which resulted in the ones included in this section of the Guide. Figure 2-9 illustrates the difference in terms of the probability of flow breakdown between unmanaged and managed motorway operation over a 15-minute period. The curves illustrate that compared to the before situation, a significant breakdown risk starts to occur at much higher flows in the after situation. This means that high traffic volumes can be managed at a much lower flow breakdown risk with managed motorway technology. Details on this comparison can be found in Section 3.8.

Figure 2-10 to Figure 2-13 show an unmanaged motorway’s flow breakdown risk (FBR) and productivity as a function of the flow rate, for 2 lane to 5 lane cross-sections for standard conditions (i.e. gradient  $\leq 2\%$  and HGV-percentage = 15%). FBR curves for other gradients or HGV-percentages can be produced by changing the flow rate (value in the x-axis) proportionally to the change in Maximum Sustainable Flow Rates as included in Table 2-9 to Table 2-12.

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3,550	3,400	3,225	3,075	2,950	2,825	2,700
3	5,150	4,925	4,700	4,475	4,300	4,125	3,925
4	6,625	6,325	6,050	5,750	5,525	5,300	5,075
5	7,875	7,525	7,200	6,850	6,575	6,300	6,025

Table 2-9: Carriageway MSFR design values (veh/h) – unmanaged motorways ( $s \leq 2\%$ )

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3,375	3,225	3,075	2,925	2,800	2,700	2,575
3	4,875	4,675	4,450	4,250	4,075	3,900	3,725
4	6,300	6,025	5,750	5,475	5,250	5,025	4,825
5	7,475	7,150	6,825	6,500	6,250	6,000	5,725

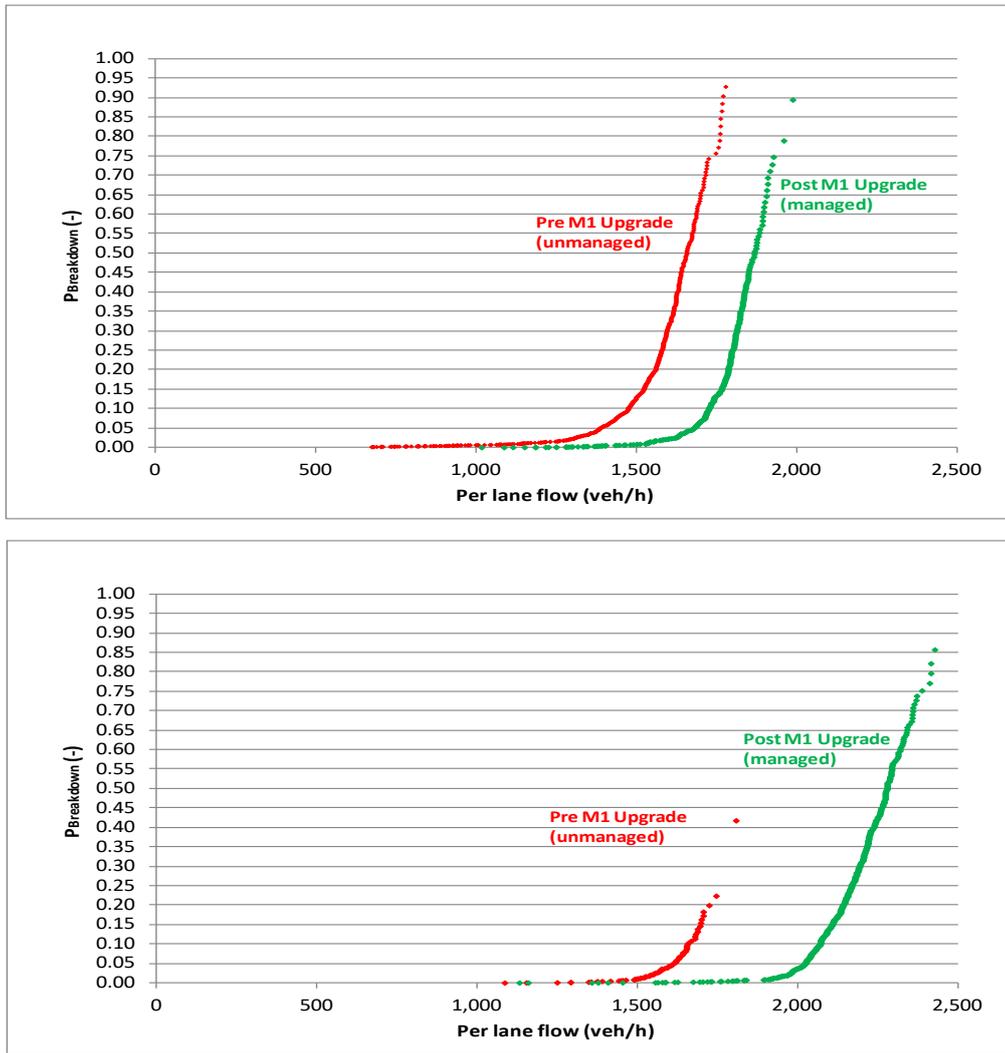
**Table 2-10: Carriageway MSFR design values (veh/h) - unmanaged motorways (2% < gradient s <= 3%)**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3,200	3,050	2,900	2,775	2,650	2,550	2,450
3	4,625	4,425	4,225	4,025	3,850	3,700	3,550
4	5,950	5,700	5,450	5,175	4,975	4,775	4,550
5	7,100	6,775	6,475	6,175	5,925	5,675	5,425

**Table 2-11: Carriageway MSFR design values (veh/h) - unmanaged motorways (3% < gradient s <= 4%)**

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	2,950	2,800	2,675	2,550	2,450	2,350	2,250
3	4,275	4,075	3,900	3,700	3,550	3,425	3,275
4	5,500	5,250	5,025	4,775	4,600	4,400	4,200
5	6,550	6,250	5,975	5,675	5,450	5,225	5,000

**Table 2-12: Carriageway MSFR design values (veh/h) - unmanaged motorways (4% < gradient s <= 5%)**



**Figure 2-9: Flow breakdown probability over 15 minutes for managed and unmanaged motorways (Warrigal Road (3 lanes, top) and Hallam Bypass (2 lanes, bottom), outbound direction)**

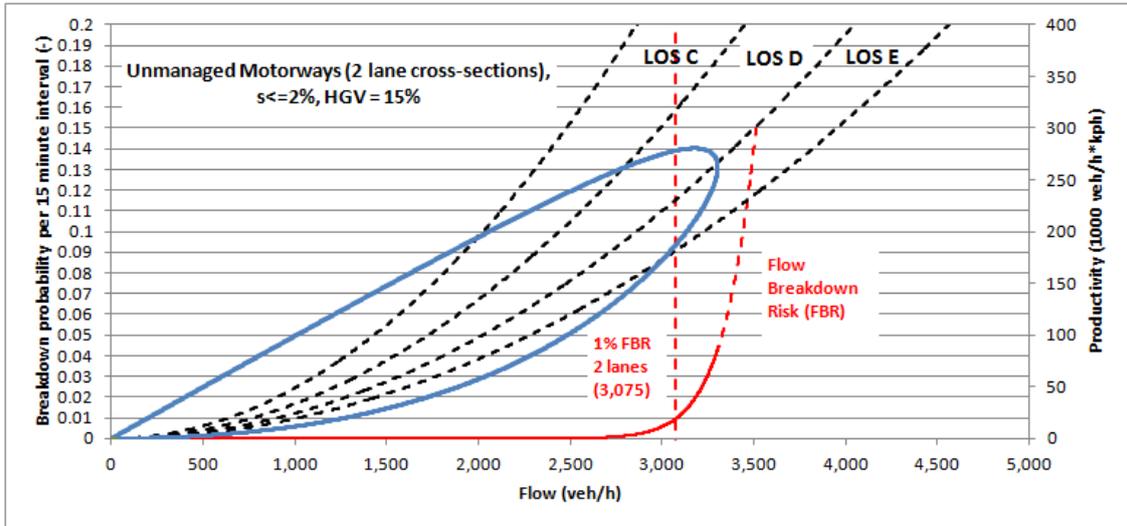


Figure 2-10: Breakdown probability and productivity plotted against flow rate (2 lane cross-sections)

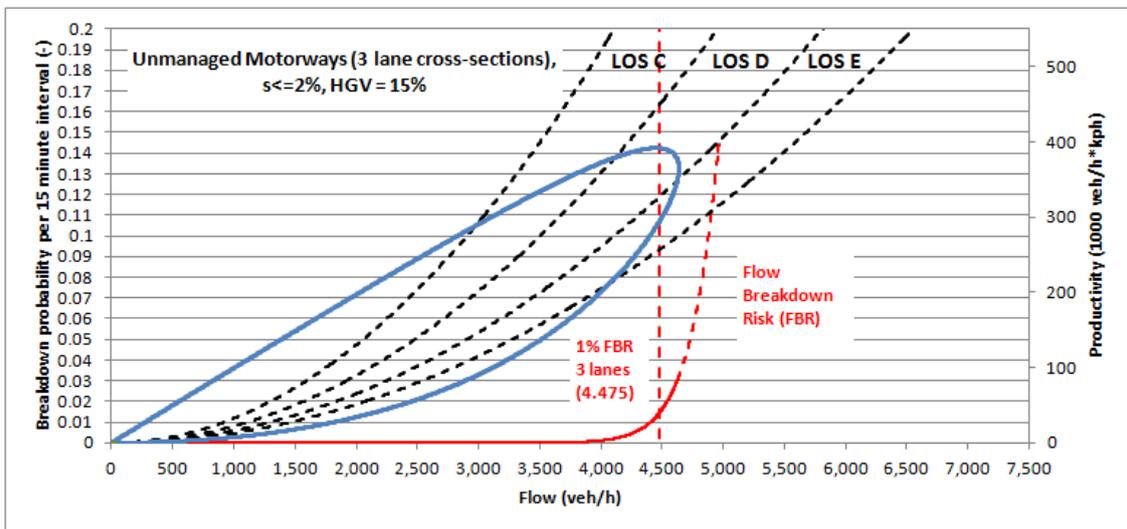


Figure 2-11: Breakdown probability and productivity plotted against flow rate (3 lane cross-sections)

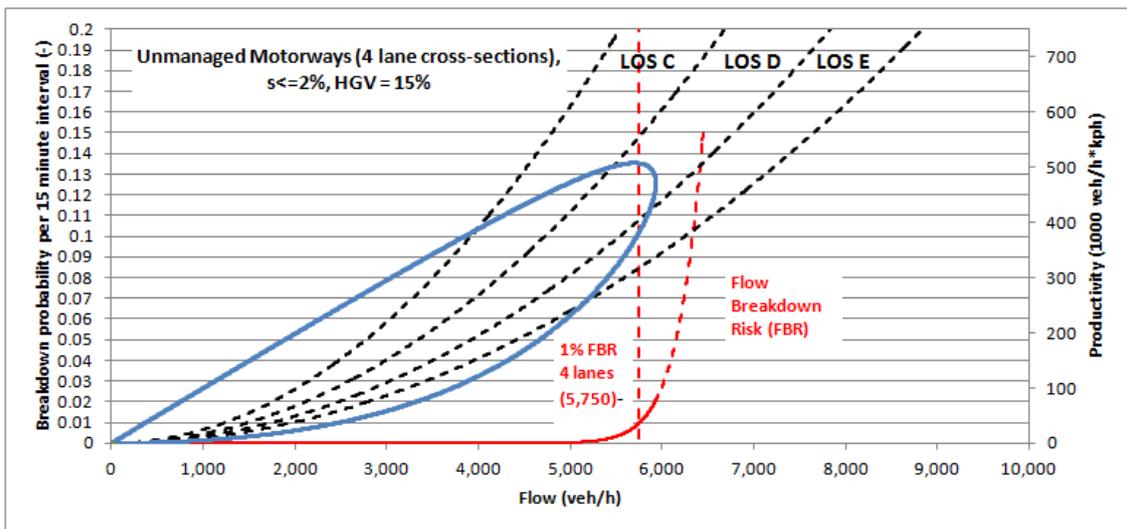


Figure 2-12: Breakdown probability and productivity plotted against flow rate (4 lane cross-sections)

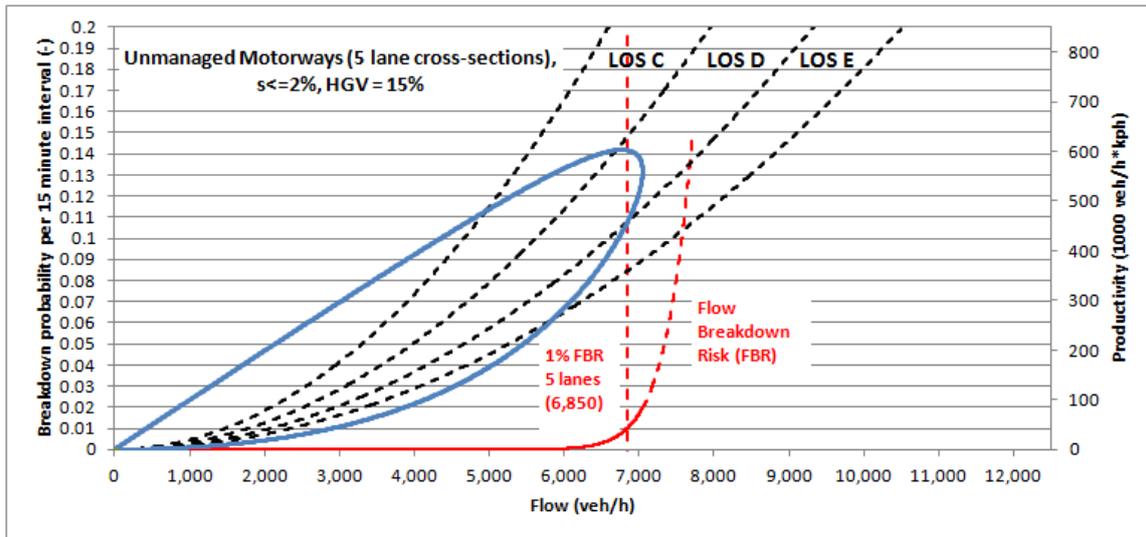


Figure 2-13: Breakdown probability and productivity plotted against flow rate (5 lane cross-sections)

## 2.6 Typical speed-flow relationship curves

Based on measured data and curves fitted to speed and volume data for individual sites, typical speed-flow relationship curves for surface managed motorway sections ( $s \leq 2\%$ , HGV = 15%) were developed.

The following procedure was applied:

- Van Aerde curve (refer to Section 3.3.2, Approach 1 - Traffic flow model) fitted for individual sites as described in Section 3.5.4;
- Parameters  $C_0$ ,  $C_1$  and  $C_2$  averaged for cross-section types (2 lane, 3 lane, 4 lane, 5 lane);
- Free flow speed assumed to be 100 km/h; and
- Curves normalised so that they represent standard conditions (e.g. end point of curve should represent maximum flow rate for standard conditions rather than those determined at measurement sites characterized by non-standard conditions).

Figure 2-14 to Figure 2-17 show the corresponding curves.

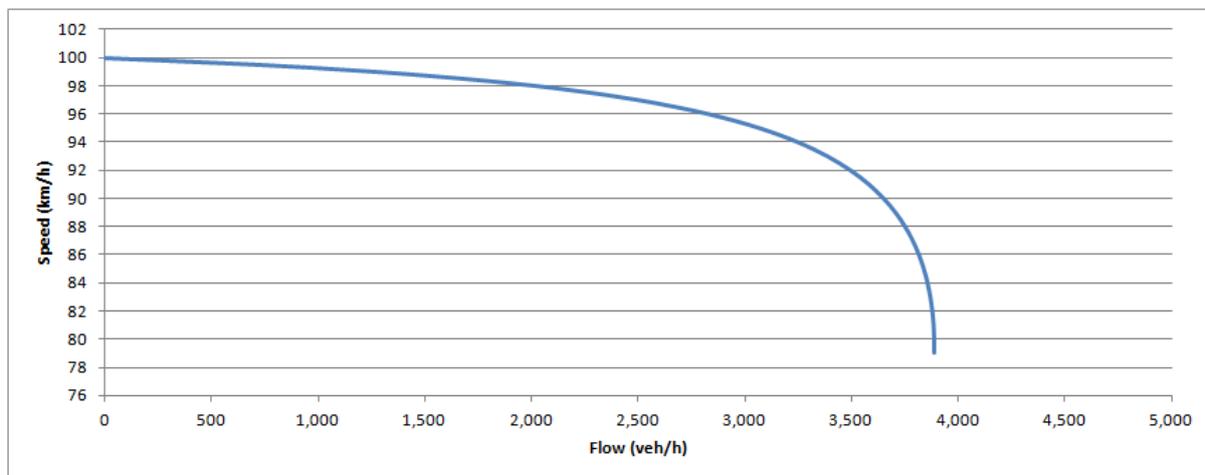
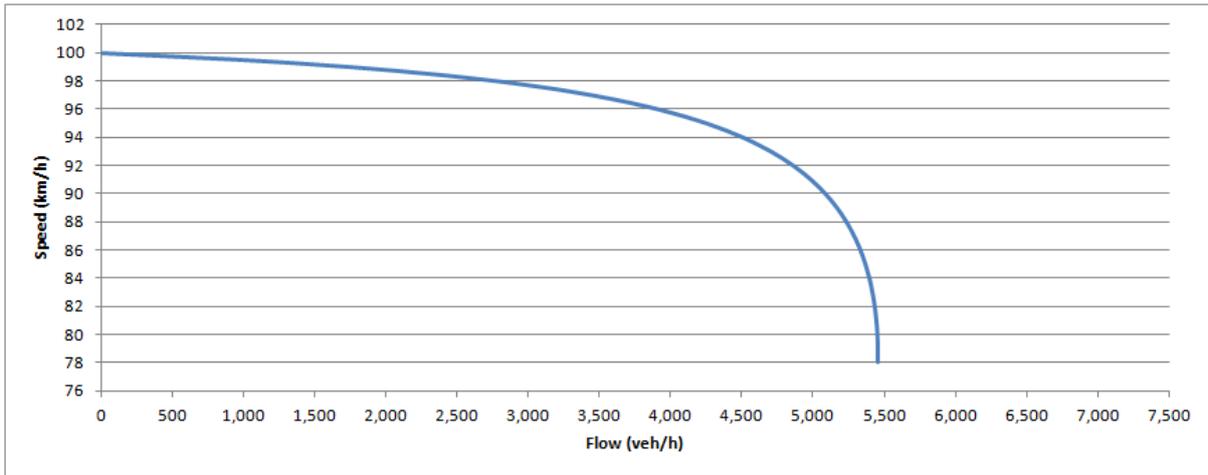
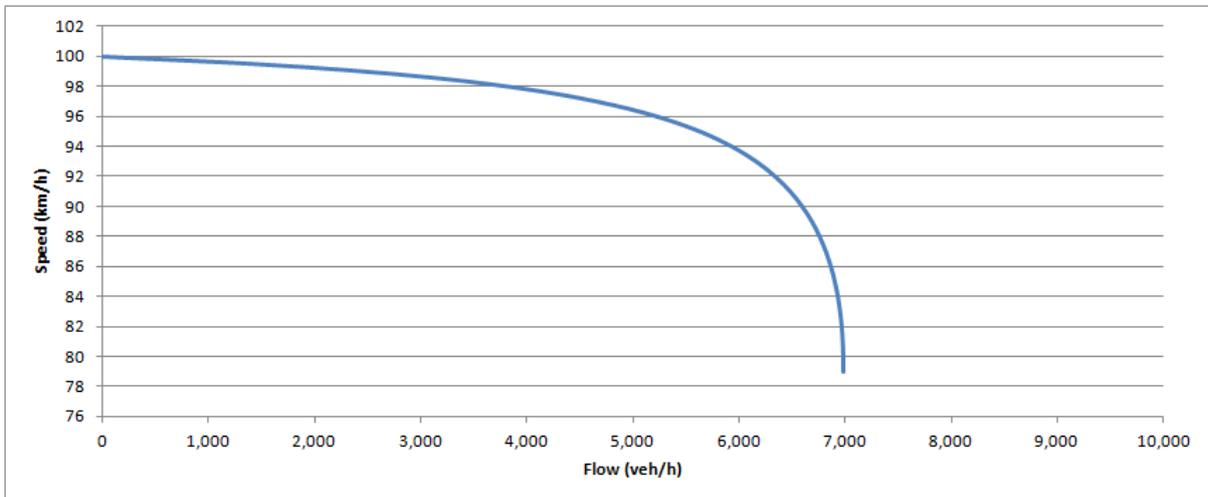


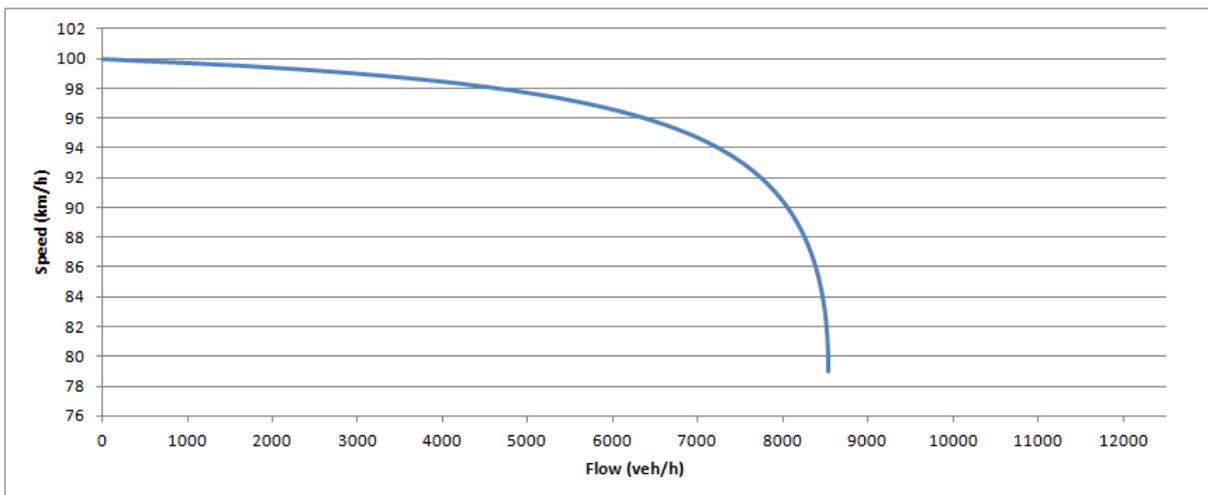
Figure 2-14: Average vehicle speed for 2 lane cross-sections (100 km/h speed limit)



**Figure 2-15: Average vehicle speed for 3 lane cross-sections (100 km/h speed limit)**



**Figure 2-16: Average vehicle speed for 4 lane cross-sections (100 km/h speed limit)**



**Figure 2-17: Average vehicle speed for 5 lane cross-sections (100 km/h speed limit)**

For potential reproduction of the corresponding curves (e.g. for use in corridor assessment/traffic modelling), Table 2-13 shows the corresponding van Aerde parameters:

	v0	C1	C2	C3
2 lanes	100	0.007767908	0.056542501	0.000124933
3 lanes	100	0.005137567	0.041798841	9.30283E-05
4 lanes	100	0.003883773	0.028289341	7.70571E-05
5 lanes	100	0.003102669	0.02306647	6.39863E-05

**Table 2-13: van Aerde model parameters for different cross-sections**

The van Aerde equation is as follows:

$$k(v) = \frac{1}{C1 + \left(\frac{C2}{v0 - v}\right) + C3 * v}$$

- Where:
- k = density (veh/km)
  - v0 = average current speed in free flow traffic (km/h)
  - C1, C2, C3 = model parameters

The solution of the van Aerde equation for speed s (km/h) as a function of flow q (veh/h) is below:

$$s(q) = \frac{1}{2} * \frac{\frac{v0}{q} + C1 - C3 * v0}{\frac{1}{q} - C3} + \sqrt{R}$$

Where:

$$R = \left( \frac{\frac{1}{2} * \frac{v0}{q} + C1 - C3 * v0}{\frac{1}{q} - C3} \right)^2 - \frac{C1 * v0 + C2}{\frac{1}{q} - C3}$$

## 2.7 Auxiliary lanes

For the purpose of planning and design, auxiliary lanes shall not be treated as through lanes, but require a reasonable assessment of the likely traffic flow conditions considering factors such as entering and exiting traffic, share of short trips entering at the start and exiting at the end of the auxiliary lane, as well as lane length. Such lanes are generally used to accommodate the significant merging and diverging movements between interchanges including the corresponding turbulences and to absorb spatial and temporal traffic concentrations. They are primarily used by vehicles either entering or exiting the motorway.

For an auxiliary lane spanning two interchanges only, Maximum Sustainable Flow Rates for the mainline shall not be increased to reflect the additional lane. Any deviation from this requires a rationale based on real-life examples and measured traffic data while being consistent with the analysis methodologies described in Section 3.3.3.

More work on the determination of Maximum Sustainable Flow Rates for auxiliary lanes, in particular complex ones spanning more than two interchanges, will be carried out in the future.

## 2.8 Tight curves

The limited analysis undertaken to date has not produced clear evidence that tight curves reduce capacity and Maximum Sustainable Flow Rates for all situations. However, since average travel speeds on tight curves decrease, there is a clear negative impact on productivity as defined in Section 1. From a traffic efficiency perspective, it is therefore not recommended to adopt curve radii smaller than a 750 metre threshold when planning and designing motorway infrastructure where flows that maximise productivity of the asset can be expected on adjoining sections, irrespective of any potential negative impacts of tight curves on capacity and Maximum Sustainable Flow Rates. Details on these considerations can be found in Section 3.9.

## 2.9 Initial application of Maximum Sustainable Flow Rates

### 2.9.1 Planning

The standardised managed motorway speed-flow relationship curves as shown in Section 2.6 (Figure 2-14 to Figure 2-17) could provide input to transport planning related tasks (e.g. corridor assessment/traffic modelling). Based on the corresponding MSFR, similar relationships can be developed for surface managed motorway sections with non-standard conditions (i.e. gradient > 2% and HGV-percentage other than 15%), tunnel sections or unmanaged motorways.

### 2.9.2 Design

Guidance on the use of MSFR for design is initially to be applied in determining the number of lanes to be considered in a motorway design process. It is noted that additional operational considerations will be required to further refine site specific requirements.

When designing a road, a balance must be achieved between construction costs and level of service or benefits. The objective of the designer is generally to achieve the desired level of service which besides the traditional US HCM LOS concept can also be defined as a certain Flow Breakdown Probability over a certain period at acceptable costs. Traffic demand usually varies widely throughout the year, and it would be uneconomic to design a road for the maximum hourly volume that could be expected. Instead, a lower volume is chosen which will be exceeded for a particular number of hours during the year.

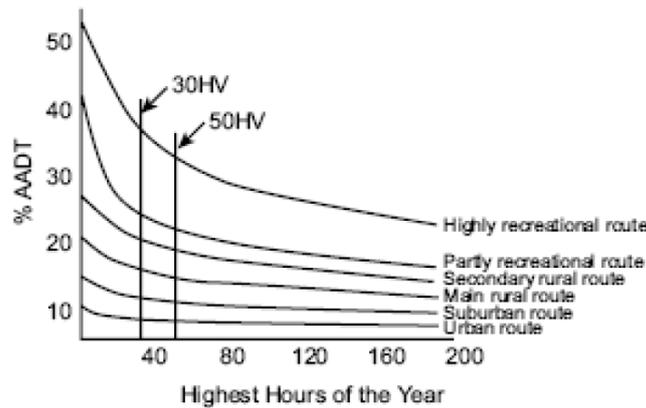
It is common practice that the 'Design hourly volume' as the forecast traffic demand results from strategic traffic modelling or is derived from traffic counts. Depending on the location or functionality of the road, the 30th or the 50th highest hourly volume (denoted as 30 HV or 50 HV) is often used while taking into consideration the traffic growth over a design period.

Where hourly volumes are available for the whole year 30 HV or 50 HV can be found directly by sorting the data. Where this is not the case, approximations of the relationship between an estimated AADT value and the n HV as shown in Figure 2-18 may be helpful.

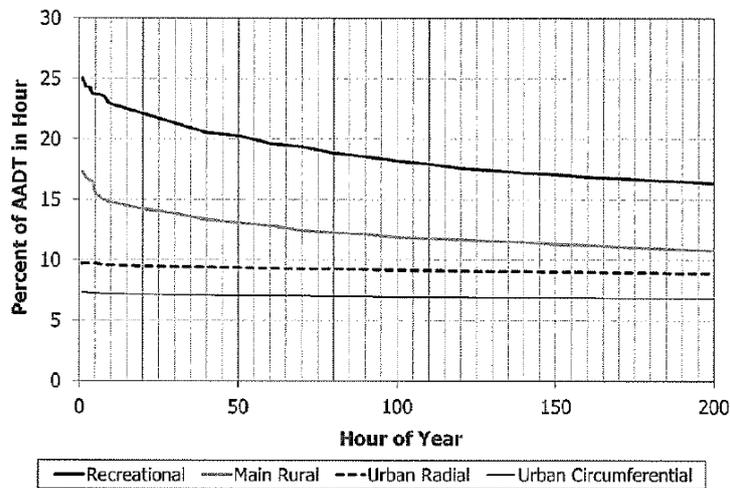
The Austroads 'Guide to Traffic Management Part 3: Traffic Studies and Analysis in Section A.4.4' provides some detailed information about the estimation of a suitable Design hourly volume from the Annual Average Daily Traffic (AADT).

For example, a metropolitan motorway corridor with an AADT of 180,000 veh/d for both directions would have an AADT of 90,000 veh/d for the analysed carriageway. Multiplying this value by a factor of around 8% ('Urban route') results in a Design hourly volume of 7,200 veh/h.

It should be noted that the curves for 'Highly recreational route' and 'Partly recreational route' in Figure 2-18 may not be realistic any more since for most of the corresponding holiday destinations travel patterns have changed over recent years. This means that more traffic outside of classical holiday periods and week-ends has been accommodated. Therefore, Figure 2-19 shows a comparable graph adopted from the US HCM 2016.



**Figure 2-18: Typical relationships between hourly volumes and AADT (Source: Austroads, 2013)**



**Figure 2-19: Typical relationships between hourly volumes and AADT (Source: HCM, 2016)**

The Design hourly volume that results from strategic traffic modelling or has been derived from traffic counts or similar as explained above shall be compared to Maximum Sustainable Flow Rates (MSFR) as listed in Table 2-1 to Table 2-12. The Design hourly volumes for individual motorway sections must not exceed these flow rates. If this cannot be achieved, the design parameters must be changed, i.e. usually the number of lanes revised. The comparison between Design hourly volume and MSFR needs to be repeated for every motorway section between two interchanges. The ‘weakest link’, i.e. the section with the highest traffic demand (or Design hourly volume) to capacity (or MSFR) ratio determines the functionality of the motorway system provided by the design (refer to Section 1.2).

The following examples explain the use of MSFR included in this Guide in the context of the corresponding procedures and examples as provided by the HCM 2016<sup>2</sup>.

**Example 1**

The determination of the number of lanes required to satisfy a certain demand volume is a classic design application. The following description goes through the needed steps and illustrates the corresponding use of MSFR stated in this Guide:

<sup>2</sup> Examples based on Chapter 26 of the HCM 2016 (from Page 26-22 onwards)



**Example 2**

The task is to determine the type of motorway facility required to satisfy a certain demand volume.

**Step 1: Input Data**

- Managed or unmanaged motorway (urban)
- Demand volume (Design hourly volume): 3,100 veh/h (one direction)
- Terrain condition:  $s \leq 2\%$
- Traffic composition (percent of heavy vehicles): 15% HGV
- Other: Not relevant (e.g. width of lanes and right-side or overall lateral clearance assumed to be consistent with Austroads design guidelines)

**Step 2: Estimate Number of Lanes Needed (iterative)**

2 lanes, unmanaged:

$$q = 3,100 > MSFR = 3,075 \text{ (2 lanes, unmanaged insufficient)}$$

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	3,550	3,400	3,225	3,075	2,950	2,825	2,700
3	5,150	4,925	4,700	4,475	4,300	4,125	3,925
4	6,625	6,325	6,050	5,750	5,525	5,300	5,075
5	7,875	7,525	7,200	6,850	6,575	6,300	6,025

2 lanes, managed (selected because iteration 1 only showed a small difference between  $q$  and MSFR):

$$q = 3,100 < MSFR = 3,625 \text{ (2 lanes, managed sufficient)}$$

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	4,175	3,975	3,800	3,625	3,475	3,325	3,200
3	6,050	5,775	5,525	5,250	5,050	4,850	4,625
4	7,800	7,450	7,125	6,775	6,500	6,225	5,975
5	9,275	8,875	8,475	8,050	7,750	7,425	7,100

Where:  $q$  = Design hourly volume (veh/h)  
 MSFR = Maximum Sustainable Flow rates as listed in Table 2-9 and Table 2-1 (veh/h)

**Step 3: Conclusion**

The appropriate type of motorway facility for the given Design hourly volume is a 2 lane managed motorway.

**Example 3**

The task is to determine the type of motorway facility required to satisfy a certain demand volume.

**Step 1: Input Data**

- Managed motorway (urban)
- Demand volume (Design hourly volume): 7,000 veh/h (one direction)
- Terrain condition:  $s \leq 2\%$
- Traffic composition (percent of heavy vehicles): 15% HGV
- Other: Not relevant (e.g. width of lanes and right-side or overall lateral clearance assumed to be consistent with Austroads design guidelines)

**Step 2: Estimate Number of Lanes Needed (iterative)**

3 lanes:

$$q = 7,000 > MSFR = 5,250 \text{ (Number of lanes insufficient)}$$

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	4,175	3,975	3,800	3,625	3,475	3,325	3,200
3	6,050	5,775	5,525	5,250	5,050	4,850	4,625
4	7,800	7,450	7,125	6,775	6,500	6,225	5,975
5	9,275	8,875	8,475	8,050	7,750	7,425	7,100

4 lanes:

$$q = 7,000 > MSFR = 6,775 \text{ (Number of lanes insufficient)}$$

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	4,175	3,975	3,800	3,625	3,475	3,325	3,200
3	6,050	5,775	5,525	5,250	5,050	4,850	4,625
4	7,800	7,450	7,125	6,775	6,500	6,225	5,975
5	9,275	8,875	8,475	8,050	7,750	7,425	7,100

5 lanes or 2 x 2 lanes:

$$q = 7,000 < MSFR = 8,050 \text{ (5 lanes sufficient)}$$

$$q = 7,000 < MSFR = 2 * 3,625 = 7,250 \text{ (2 carriageways with 2 lanes each sufficient)}$$

Number of lanes	HGV Percentage (%)						
	0%	5%	10%	15%	20%	25%	30%
2	4,175	3,975	3,800	3,625	3,475	3,325	3,200
3	6,050	5,775	5,525	5,250	5,050	4,850	4,625
4	7,800	7,450	7,125	6,775	6,500	6,225	5,975
5	9,275	8,875	8,475	8,050	7,750	7,425	7,100

Where:  $q$  = Design hourly volume (veh/h)

$MSFR$  = Maximum Sustainable Flow rates as listed in Table 2-1 (veh/h);

**Step 3: Conclusion**

The appropriate type of motorway facility that could be considered for the given Design hourly volume is either a 5 lane managed motorway or two 2 lane managed motorways (compare to Figure 2-5).

## Chapter 3: Details on the definitions, methodologies and analysis used

**Chapter 3** contains details on the 'capacity', productivity and flow breakdown probability determination methodologies including on the corresponding measurement results and other scientific considerations.

**Chapter 1** includes details on the reasoning behind the adoption of Maximum Sustainable Flow Rates to plan, design and manage motorways, and

**Chapter 2** provides guidance on the Maximum Sustainable Flow Rates for planning and design of motorways.





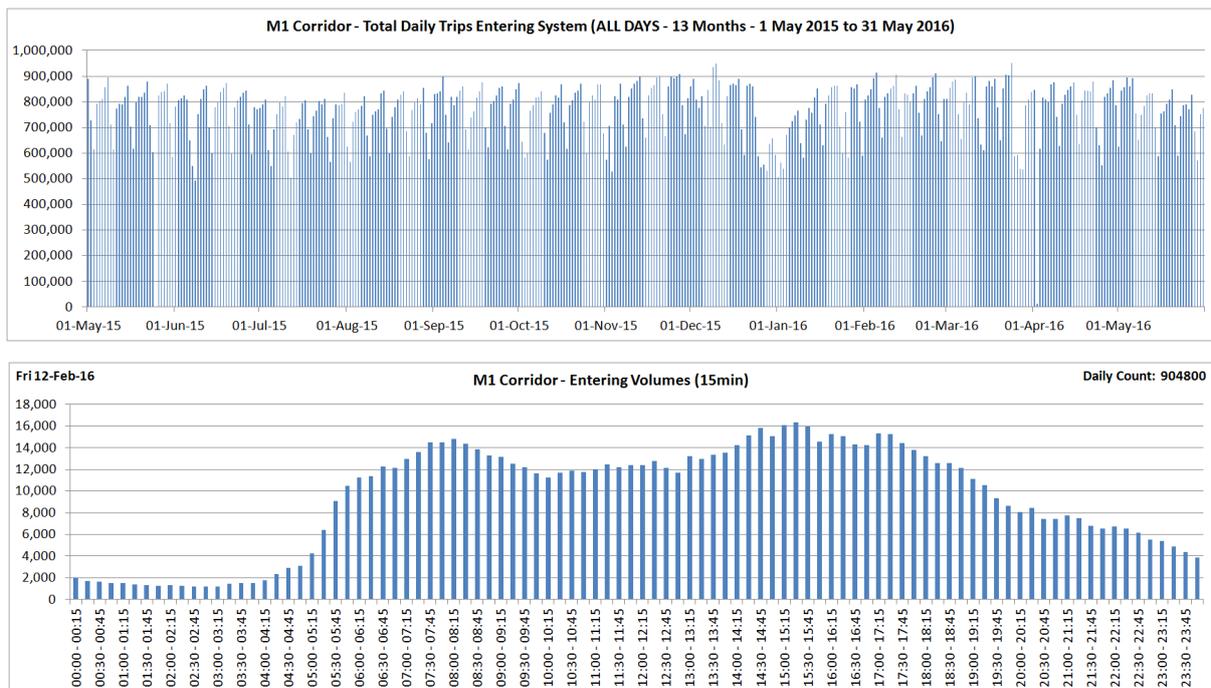
### 3 Definitions, methodologies and analysis

#### 3.1 Need for the Guide

In 2016 the Metropolitan Melbourne motorway network represented 7% of the urban arterial road lane kilometres, yet carried 40% of the urban arterial road travel. Reliance on the motorway network to provide arterial road travel is trending upwards, with a 50% share likely in the relatively near future.

The current share of motorways in all urban arterial road travel has doubled in the last 15 years. Should similar economic circumstances continue, the traffic volume on the motorway network is likely to grow at a similar rate over the next decade, predominantly through significant peak period spreading because ‘capacity’ constraints are limiting the growth during current peak periods. Although some additional motorway ‘capacity’ will be added, the motorway network share is likely to remain at around 7% as the urban arterial road network is also growing as new suburbs are being developed and some existing roads are upgraded with additional lanes.

Along a major motorway such as Melbourne’s M1 Corridor with a length of approximately 70 kilometres, more than 1 million journeys a day are serviced during a peak period (vehicle occupancy assumed to be 1.2 persons per vehicle).



**Figure 3-1: Vehicle trips along the M1 Corridor**

The determination of realistic motorway maximum sustainable flow rates and speed flow relationship curves is essential for proper project planning, business case development, road design, traffic engineering as well as for the real-time operations of motorway networks. Understanding of such flow rates and operational curves will ultimately lead to outcomes such as increased traffic flow efficiency and productivity, improved safety and environmental benefits.

However, in practice, motorway ‘capacity’ is often determined without full consideration of all the operational factors which cause ‘random fluctuations’, resulting in disruptions to the traffic flow. Such approaches do not reflect the likely operational outcomes that the community will experience when new motorways are built or upgraded, particularly in a major urban environment where demand will never be fully catered for. Hence the need to determine Maximum Sustainable Flow Rates relating to

probability of flow breakdown and maximum productivity that realistically reflect the on-road outcomes achieved. Their determination must utilise appropriate robust and repeatable statistical analysis methods using measured traffic data from a wide range of demand and traffic flow patterns. Maximum Sustainable Flow Rates that relate to probability of flow breakdown and maximum productivity reflect motorists' experience, community expectations and the realised economic benefit of the built motorway system.

It is important to note that international evidence is emerging that a well-planned, built and operated motorway (i.e. one where traffic flows well) is also a safe motorway. Under congested conditions, the crash rate has been shown internationally to be 5-6 times higher than in the free flow state. Motorways in general have a very good safety record from an overall crash rate perspective. However, given the huge number of vehicle kilometres travelled on motorways, it is apparent that the absolute number of crashes occurring on them is still very high. Therefore, any potential safety improvement on motorways offers great potential for the improvement of overall road safety.

It has become increasingly evident that the US Highway Capacity Manual (Transportation Research Board, USA Highway Capacity Manual, 2016), which is primarily referenced by Australian practitioners and is also heavily cross referenced in Australian guidelines (i.e. Austroads publications), provides 'capacity' values that do not always reflect the Maximum Sustainable Flow Rates. Potential alternative guidelines from other countries which were reviewed for this Guide (e.g. UK, Germany or The Netherlands or Sweden) are also not widely used in Australia.

'Capacity' values and design methodologies included in the different guidelines show significant differences. For example, the report 'Comparison of Key Freeway Capacity Parameters on North American Freeways with German Autobahns Equipped with a Variable Speed Limit System' (Bertini et al., 2006) shows that there are quite significant differences between the US HCM 2000 and the 'German HCM' 2001. For a three lane cross-section on level terrain with a free flow speed of 100 km/h and 10% trucks, the US HCM determines a 'capacity' of around 2210 veh/h/ln (equivalent to 2320 pc/h/ln) compared with a 'German HCM' value of 1950 veh/h/ln. This is equivalent to a significant difference of 11.8%. Whilst it is acknowledged that these manuals have been revised since this 2005 report (i.e. US HCM in 2016 and 'German HCM' in 2015), the following significant differences still remain in the updated handbooks:

- The US HCM prescribes 'capacity' values that are generally much higher than adopted in Europe, including in the UK;
- the 'German HCM' and the 'Dutch HCM' recognise that the number of lanes affect lane 'capacity', i.e. a decreasing lane 'capacity' with an increasing number of lanes - in contrast, the US HCM implies that the number of lanes does not significantly affect lane 'capacity'; and
- the 'German HCM' which is informed by the objective of productivity maximisation recommends design for 0.9 times the nominal hourly 'capacity' value, while the 'Dutch HCM' recommends design for 0.8 times the median of the (probabilistic) 5-minute cumulative 'capacity' distribution function.

Following the review of international practice, VicRoads concluded that due to the significant differences between the different guidelines and 'capacity' values in particular, combined with the unique Melbourne (managed motorway) situation, further investigation of Melbourne's motorway performance was warranted.

## 3.2 Scope of the Guide

### 3.2.1 Motorways within large metropolitan cities

The focus of this Guide has been intentionally narrowed to only examine motorways within large metropolitan cities with very high traffic demands spread over extended peak periods which usually lead to regular occurrence of congestion. Bottlenecks typically form simultaneously at numerous locations, as experienced in urban areas in Australia, Canada, New Zealand and the USA, where demand usually exceeds the 'practical capacity' or Maximum Sustainable Flow Rate for considerable periods of the day.

Typical characteristics of such motorways include some or all of the following:

- Servicing higher populations (e.g. greater than 2 million people) within large sprawling urban areas (>50km) with relatively low density and considerable spread of suburbs;
- operating in an affluent economy that has high levels of vehicle ownership and usage, and a high consumption of goods and services;
- considerable dispersion of employment areas, activity centres, access to key sea and air ports and freight hubs etc.;
- operating in an economy that is principally services based rather than (more centralised) manufacturing based and which generates considerable levels of and widespread dispersed travel demand, i.e. couriers, spare parts delivery, trades including in-home services and tourism etc.;
- having extended peak periods every weekday where demand exceeds 'capacity' for more than 2 hours a day in a single direction;
- having many closely spaced (3-4 km) bottleneck sections during peak periods that exceed 'capacity' with regular occurrence of flow breakdown;
- having network wide traffic loading conditions such that multiple points on a motorway route or network of motorways are tending to simultaneously reach 'capacity' (or within a 5-15 min period) and hence the 'capacity' of a section of motorway is heavily influenced by the prevailing upstream and downstream conditions;
- having relatively closely spaced motorway interchanges for access and egress (typically 1-3km spacing);
- having a high percentage of trip length which are relatively short (i.e. 50<sup>th</sup> Percentile of trip length is equal to approximately 15km);
- having a high turnover of trips during peak periods with many consecutive heavily utilised access and egress points operating longitudinally along a motorway route (i.e. >1000veh/h leaving at the exit ramp followed immediately by >1000veh/h entering at the same interchange);
- having a high off peak period during the middle of every weekday having some motorway sections operating near or at 'capacity' (>80% of 'capacity' including on major bridges, tunnels and critical motorway segments, e.g. West Gate Freeway from the West Gate Bridge to City Link Tunnels and Hallam Bypass);
- traffic loadings such that multiple factors are impacting simultaneously on the motorway segment such as constantly changing vehicle mix, narrow lanes, narrow shoulders, variable Origin-Destination (OD) patterns, weaving, merging and lane changing required to achieve lane balance; and
- having numerous traffic control and information systems affecting 'capacity' and traffic patterns.

### 3.2.2 Maximum sustainable flow rates for metropolitan managed motorways

Australia is relatively unique in that it has a maximum urban motorway speed limit of 100km/h combined with an active enforcement regime as a result of long standing road safety programs. In the State of Victoria the tolerance on speed limit is small (+3km/h). This factor appears to result in less differential speeds and provides some additional traffic flow stability when demands approach maximum observable flows which may not be seen in other international cities where this regime is not applied.

The definition of a Managed Motorway internationally and across Australia is very wide and includes many broad concepts. For example, any motorway having one or more ITS tools operating (e.g. VMS, VSL, basic lane control systems or ramp metering) may be referred to as a Managed Motorway; hence there can be considerable confusion about what constitutes an effective managed motorway.

This Guide provides direction for Managed Motorways that are characterised by the following features:

1. Physical design (geometric) of the motorway to ensure safe operation and to minimise turbulence in the traffic flow on the mainline and ensure the entry and exit ramps are designed to store and regulate dynamic traffic demands into and out of the motorway network;
2. operational design to ensure the motorway can be actively managed in real-time and can be controlled during non-recurrent and incident events, supported by the provision of lane control, speed control and messaging to support safe and efficient operations;
3. real-time feedback control through adoption of control algorithms and ITS tools to operate at the lane, link, route and network levels; these algorithms and tools enable the control system to activate and adapt in a targeted manner to the many instantaneous traffic flow problems that arise in a motorway network when traffic conditions put it under stress;
4. a consistently applied organisation operational policy (written and/or practiced) that recognises that the best overall outcome for the entire road network requires motorways to be managed to sustain high safety and productivity outcomes at all times;
5. high quality ITS devices deployed in the roadway (incl. detection, telecommunication, power and control systems) with a strong focus on high accuracy and rapid delivery of real-time data;
6. the quality and timeliness of the day to day (24/7) maintenance service for all ITS devices ensuring high availability of ITS tools enabling their full functional capabilities;
7. the quality and functionality of the ITS tools deployed in the Traffic Management Centre (TMC) and by specialist network optimisation teams (e.g. user interface and backend system);
8. the focus on real-time operation of the road including training of staff;
9. the focus on regular historical analysis of motorway performance;
10. the focus on regular tuning of the system parameters by specialist optimisation teams with the aim of seeking improved performance understanding, to enable continuous improvement; and
11. the production and maintenance of high quality practice oriented design guides for use by practitioners (with regular reviews of guides to incorporate new and emerging learnings).

A particular focus of Managed Motorways is to manage operations to avoid the motorway flow and speed reductions caused by flow breakdowns, i.e. the transition from 'free flow' to 'forced flow' which coincides with a significant loss in productivity (i.e. traffic speed and traffic flow).

A motorway can only achieve its highest productivity if both appropriate design and appropriate operational practices are implemented. Hence the definition of a Managed Motorway used in this Guide implies that the following four criteria are all met:

- **an advanced system wide coordinated ramp metering algorithm** is deployed that can determine real-time 'operational capacity' and smooth and balance demand across the network to maximise mainline productivity and also achieve real-time flow recovery after traffic flow has broken down;
- **all managed motorway sections cater for both ramp storage and discharge requirements**, i.e. they have the required number of contiguous upstream ramp meters which have been designed to appropriately manage demand and are controlled within a coordinated system whose policy objectives are to maximise the productivity of the motorway network;
- **the 'design capacity' of a managed motorway operation is determined by its weakest link** (refer US 'HCM 2016' and 'German HCM') - therefore any uncontrolled entry above a certain threshold (e.g. 200 veh/h, refer to (VicRoads, 2013), Table 6.1) results in that section of road being deemed to be unmanaged; and
- if the operational policy or the corresponding management processes allow any excess ramp queues to be released inappropriately into the motorway under normal operations (i.e. the operational regime generates poorly controlled on-ramps) and hence creates avoidable temporary bottlenecks, the motorway is not considered to be managed - hence a 15% reduction needs to be applied to any 'capacity' or Maximum Sustainable Flow Rate and this must apply to all sections downstream of the uncontrolled entry.

### 3.2.3 Caveat

The deployment of advanced managed motorway technologies in Victoria has been demonstrated to significantly influence motorway 'capacity' outcomes over extended periods where peak hour demand typically exceeds 'capacity'. The continuing advancement of technologies and changes in how the network is used (i.e. the likely gradual change in vehicle types and vehicle mix, the deployment of semi-autonomous or autonomous vehicles and the future need for autonomous transport networks that can optimise networks rather than just optimise for individual trips) may influence 'capacity' outcomes at some point in the future.

To account for the advancement of technologies, the following approaches are recommended:

- Since automation will increasingly impact on 'capacity' (one way or the other) and exact timelines are unknown (note that predictions vary widely), maximum sustainable flow rates (MSFR) presented here should be reviewed and updated at regular intervals, e.g. every 5 years. This should be based on the selected measurement sites and MSFR determination methodology presented here; and
- All traffic modelling, including underlying 'capacity' assumptions should be based on measured headway data distributions which cover car following as well as lane changing activity. These influences are implicitly included in the Maximum Sustainable Flow Rates and speed-flow-relationships stated in this Guide. Many models ignore the 'friction' between lanes, i.e. they don't consider the slowing effect due to drivers looking for opportunities to change lanes and the corresponding impact on 'capacity'. At the time of writing, at two test sites on the Monash Freeway (4 lane cross-sections), more than 2,000 lane changes per kilometre for each hour are being measured when the motorway is operating near 'capacity'. Under Metropolitan Melbourne motorway network conditions with a relatively short trip length and a high vehicle turnover rate (on average every vehicle that is in the motorway system gets replaced by another vehicle after 15 km), very few lane changes are discretionary but rather are needed to fill all lanes to achieve 'capacity'.

## 3.3 Measurement methodology

### 3.3.1 Data

In deriving a methodology that reflects the HCM definition (refer to Section 1.3, the 'capacity' values and Maximum Sustainable Flow Rates developed by VicRoads have been measured at active **bottlenecks** and were determined on the basis of **statistical analysis** of data to capture the effects of **systematic** and **random** fluctuations under a wide range of **demand patterns**. The following definitions are used:

- **Bottlenecks** are characterised by high demand and/or low 'capacity', which as a combination leads to observed loss of performance on a regular basis, e.g. a speed drop and/or flow drop resulting in a significant reduction in productivity. 'Capacity' can only be measured in bottlenecks and they determine the overall 'motorway facility' performance capability. A bottleneck can be a geometric constraint and/or can be due to operational influences. It is the weakest link of a corridor or a part of it. Other cross-sections may have higher capacities (e.g. due to less lane changing activity). However, these are less relevant because they don't determine corridor 'capacity'. A managed motorway will always have a number of bottlenecks since it is operated so that throughput and/or productivity gets maximised, i.e. there is enough traffic demand in the system (including on the on-ramps) to fill it.
- **Statistical analysis** requires a recognised and repeatable statistical methodology measuring over a sufficient number of days which include a high proportion of days with bottleneck activations. Analysis of Metropolitan Melbourne data showed that data measured over 1 month allows for reproducible 'capacity' values within narrow margins if based on data from a different but equally long time period.
- **Systematic:** cyclic patterns influenced by seasonal, monthly and weekly variations, daylight/darkness, wet/dry etc.
- **Random:** influenced by unplanned events, minor incidents and traffic disturbances (generally going unreported and causing minor disturbances to lane flows), driver behaviour etc.
- **Demand Patterns** vary from minute to minute, hour to hour, day to day, week to week, season to season and section to section and include changes in average trip lengths, Origin - Destination (OD) patterns and vehicle mix combinations.

'Capacity' and the other related parameters (Maximum Sustainable Flow Rate, productivity, probability of flow breakdown) vary over time because while some factors are relatively static (refer to Table 1-2), there are also a number of dynamic factors involved (refer to Table 1-3). For example, plotting the fundamental relationships on a daily basis clearly shows that different curve shapes occur every day. As a consequence, the determination of 'capacity' and other related parameters should be based on long-term measurements to get a reasonable representation of all different possible traffic states (i.e. combinations of dynamic factors).

### 3.3.2 Methodology selection

The following paragraph describes and compares different methodologies for the estimation of 'capacity' values and Maximum Sustainable Flow Rates. Details on their application can be found in Section 3.3.3.

Some requirements for the 'capacity' analysis and therefore also for the 'capacity' determination methodology were as follows:

- Allows for the realistic comparison of 'capacity' values and Maximum Sustainable Flow Rates with forecast hourly demand values.
- As far as possible directly uses measured data rather than relies on a traffic flow model (i.e. limitation of the influence of models used/curves fitted to the data).

**Approach 1 - Traffic flow model**

- 'Capacity' is determined as the maximum flow rate of the fitted carriageway speed-density and resulting speed-flow curve according to van Aerde (van Aerde, 1995), which is the only known model that enables a realistic description of all traffic states/areas in the speed-flow diagram with a continuous curve. Other models are for example listed in (Sajjadi et al., 2014).
  - o The methodology uses hourly flow rates (as hourly 'capacity' values will get compared to hourly demand values – e.g. sourced from a Network Model such as VITM etc).
  - o A van Aerde curve is fitted to speed-density measured values (i.e. minimisation of the sum of error squares derived from a comparison between modelled (fitted curve) and measured  $k$  (density) values); measured values are based on the calculation of average  $v$  (speed) values for each density class/bin.
  - o The van Aerde equation is as follows:

$$k(v) = \frac{1}{c_1 + \left(\frac{c_2}{v_0 - v}\right) + c_3 \cdot v}$$

Equation (1):

where  $k$  = Density (veh/km) $v_0$  = Average vehicle speed when flow approaches zero (km/h) $c_1, c_2, c_3$  = Model parameters

- o Flow and hence 'capacity' as the highest flow rate is determined as the mathematical product of speed and density.

Equation (2):  $q(k, v) = k * v$

where  $q$  = Flow rate (veh/h) $k$  = Density (veh/km) $v$  = Average vehicle speed (km/h)

- o Geistefeldt suggested to exclude 1-hour intervals with unsteady flow conditions from the 'capacity' analysis (Geistefeldt, 2016) and applied this approach when verifying/updating 'capacity' values for the new 'German HCM' (FGSV, 2015). This was done because such intervals 'may represent a traffic state that never existed in real traffic flow'; this is not considered useful for Melbourne managed motorway conditions as such conditions are a regular occurrence and it would exclude too many measured values and lead to a systematic over-estimation of 'capacity'.
- Approach is suitable if
  - o Cross-section is frequently oversaturated (i.e. a bottleneck).
  - o Curve is reflective of measurements (i.e. a good fit in all areas – 'free flow', 'forced flow' and transition area).
- Advantages of this approach:
  - o Direct comparison of hourly demand values with hourly 'capacity' values possible - no (potentially problematic) transformation of 'capacity' values determined based on 5 minute or 15 minute interval data to 1 hour 'capacity' values needed.
  - o Reflects all traffic states (i.e. 'free flow', 'forced flow' and transition area).
  - o Has been used for the 'German HCM' (allows for comparison if applied in exactly the same way and for a plausibility check if applied in a slightly different way).
- Disadvantages of this approach:
  - o Measured values impact on the result; e.g. if traffic demand goes up very steeply in the morning and breakdown occurs shortly thereafter, hourly 'capacity' values partly reflect demand only (less of a risk when using rolling 1-hour averages).
  - o No transparent methodology to transform 'capacity' values into suitable design values.

## Approach 2

### Variant a)

- Methodology according to (Brilon et al., 2005), first published in (Brilon & Zurlinden, 2003)
  - o Details are described in the article 'Reliability of Freeway Traffic Flow: A stochastic Concept of Capacity' by Brilon et al.
  - o This methodology aims at estimating the 'Likelihood that it breaks down at a flow rate higher than q' and correspondingly the 'Likelihood that it breaks down at a flow rate lower than q' (which is simply 1- 'Likelihood that it breaks down at a flow rate higher than q').
  - o The 'Likelihood that it breaks down at a flow rate higher than q' is equivalent to the likelihood that it does not break down at any flow rate  $q_i$  lower than q which can be easily estimated by comparing the number of breakdowns at  $q_i$  to the total number of intervals with a flow rate equal to or higher than  $q_i$ . A detailed description of the application of this methodology is included in Section 3.3.3.
  - o 'Likelihood that it breaks down at a flow rate lower than q' over 5 minutes at a particular flow rate (traffic volume) is equivalent to the value of the cumulative 'capacity' distribution function  $F_C(q)$  for that flow rate (refer to diagrams in Section 3.5.4)
  - o A certain percentile (e.g. median) of the 5-minute-interval cumulative 'capacity' distribution function is regarded as 'capacity'.
  - o For design purposes, the forecast demand (usually hourly value) gets compared to a multiple of this 'capacity' value (e.g. median multiplied by 0.8 used for the 'Dutch HCM').
- Advantages of this approach:
  - o Is the only known methodology that results in a 'capacity' distribution that cannot be compromised by low traffic demand. This is for the following reasons:
    - Steep rise in demand in the morning peak hour is not an issue because of shorter interval lengths.
    - Cumulative 'capacity' distribution function (CDF) less well-defined in the 'upper range' if only few high flow rate intervals observed; well defined if frequently oversaturated (compare to diagrams in Section 3.5.4).
    - Consideration of censored and uncensored data representing the 'capacity' of all 5-minute-intervals:
      - Uncensored data: 'Capacity' has been reached when flow breaks down in the next interval.
      - Censored data: 'Capacity' is higher than currently measured flow rate when flow does not break down in the next interval. This information is used for the determination of the cumulative 'capacity' distribution function
    - Has been used for the 'Dutch HCM' (allows for comparison if applied in exactly the same way and for a plausibility check if applied in a slightly different way).
  - o No traffic flow model with the corresponding uncertainties around selecting the 'right' model and parameters needed (if only using the non-parametric Product Limit Method).
  - o Suitable design values or Maximum Sustainable Flow Rates can be determined based on the tolerable breakdown risk over a certain time period (compare to 'Discussion and conclusion' below)
  - o Compared to a similar methodology which is now the recommended HCM approach for capacity estimation (refer to HCM 2016, Exhibit 26-13), the following is regarded as advantageous: Cumulative 'capacity' distribution function (CDF) is relatively steady (even when using the non-parametric approach and not approximating it by a Weibull curve), easier handling (no definition of 'Flow Rate Bins' needed), result is independent of the definition of bins.
- Disadvantages of this approach:
  - o Transformation of 5 minute or 15 minute 'capacity' values to 1 hour 'capacity' values needed (if 'capacity' relevant for the design process)

- Does not reflect all traffic states (i.e. only 'free flow') which is not entirely reflective of current managed motorway operation

Variant b)

- Percentile (e.g. median) of 15-minute-interval cumulative 'capacity' distribution function (remainder as described under Variant a))

Compared to the 5-minute interval approach, the transformation into hourly values is less problematic; application over a 1 week period showed that the 'capacity' distribution function is similar to the one for 5-minute intervals.

**Discussion and conclusion**

Approach 1 is a traditional way to determine 'capacity' values. It allows for easy comparison with the majority of other established guidelines. It is also easy to embed hourly 'capacity' values into the current planning and design process as these values need to be compared to forecast hourly demand values. However, it is not suitable for the transparent determination of suitable design values or Maximum Sustainable Flow Rates which should in particular be linked to an acceptable probability of a flow breakdown. This approach is considered appropriate for plausibility checks.

Approach 2 is selected as the primary methodology to determine Maximum Sustainable Flow Rates as it allows for direct insights into the probability of a flow breakdown. Elefteriadou and Lertworawanich (Elefteriadou and Lertworanawich, 2003) recommend 'that the breakdown flow be used' to determine 'capacity', in particular because 'it is consistent with the current implication that 'capacity' is the boundary between non-congested and congested conditions'.

The stated disadvantage that 5-minute interval (or 15-minute interval) values need to be transformed into hourly values is not seen as an issue as the risk based approach does not require such a conversion. The procedure should start with an acceptable flow breakdown risk over a certain peak period (e.g. 10% over a 3 hr peak period) and acceptable hourly as well as 15-minute interval sustainable flow rates can be derived from this. An example is below:

- Acceptable breakdown risk over a 3 hr peak interval:

$$P_{Breakdown}(3h) = 10\% = 0.1$$

- Acceptable breakdown risk over a 1 hr peak interval:

$$P_{Breakdown}(1h) = 100 * (1 - \sqrt[3]{(1 - P_{Breakdown}(3h))}) = 3.5\%$$

- Acceptable breakdown risk over 15 minute intervals:

$$P_{Breakdown}(15min) = 100 * (1 - \sqrt[12]{(1 - P_{Breakdown}(3h))}) = 0.9\%$$

Approach 1 and Approach 2 (Variant b)) were applied to the same measurement sites and the results were subsequently compared (refer Section 3.3.3).

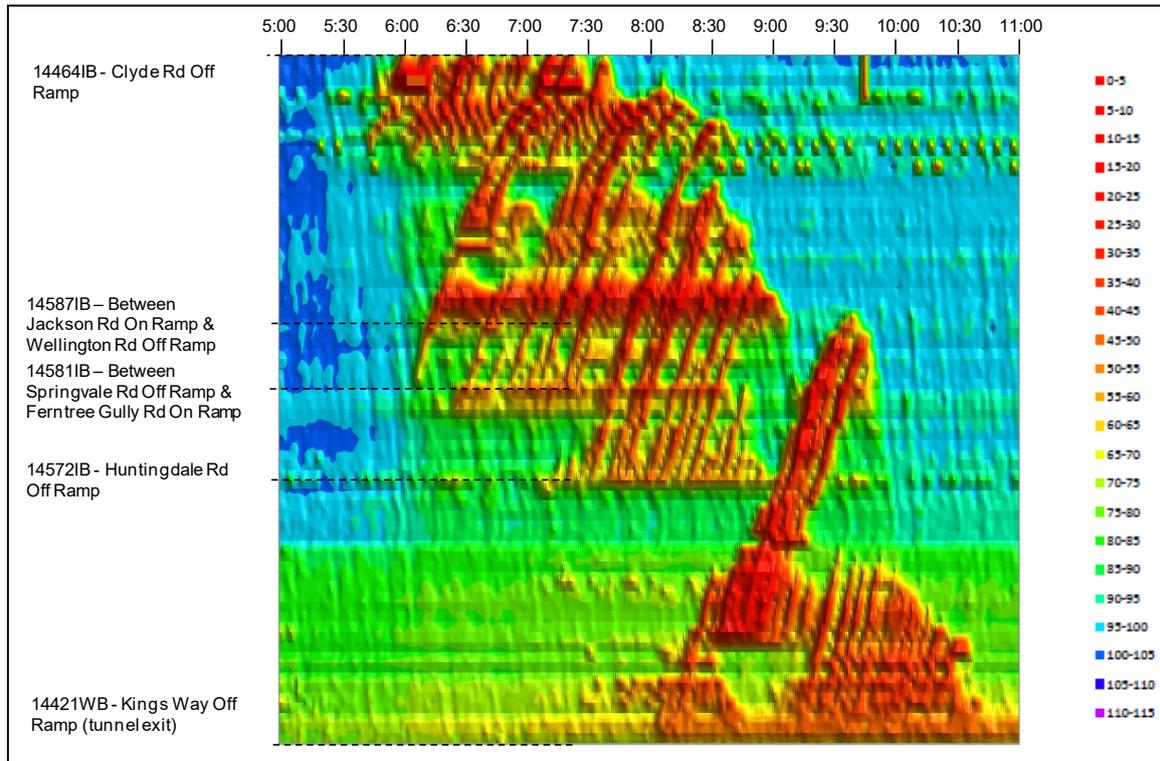
**3.3.3 Methodology application**

The following paragraph describes the application of the described methodologies to determine 'capacity' and productivity values as well as probabilities of flow breakdown as documented in this Guide.

'Capacity' can be best observed in locations and at times where traffic regularly breaks down. 'Heat Plots' as shown in Figure 3-2 were used to identify bottlenecks characterised by relatively low speeds over the day. These are often the origins of 'wide moving jams' that propagate against the direction of

travel and impact on traffic flow at upstream cross-sections. However, they are even more often the sinks of congestion or smaller perturbations on a lane level, i.e. congestion settles in such bottlenecks.

In case of the example in Figure 3-2 congestion first occurred between 6:00 AM and 6.15 AM between the Springvale Rd Off Ramp and the Ferntree Gully Rd On Ramp and settled between the Jackson Rd On Ramp and the Wellington Rd Off Ramp.



**Figure 3-2: 'Heat Plot' from the Monash Freeway (inbound speed profile)**

In order to determine the 'capacity' of the selected bottlenecks (and in line with international best practice), the influence of congestion spilling back from downstream sections was excluded by eliminating those (rolling) 1 hour or actual 15 minute intervals where the speed at the next downstream cross-section was lower than the threshold speed between 'free flow' and 'forced flow', typically 65 km/h.

Eliminating such intervals has the following effects:

- The systematically lower 'capacity' within 'wide moving jams' which is unrelated to the 'capacity' of the bottleneck itself does not influence the 'capacity' measurement at the bottleneck; and
- the influence of major incidents happening downstream of the bottleneck on 'capacity' measurement in the bottleneck is largely eliminated.

Other recognised influences on the quality of traffic flow on motorways and hence on the result of any 'capacity' analysis include changeable weather conditions, day time/night time conditions (or the corresponding light/visibility conditions) and minor incidents.

The adopted methodology does not distinguish between dry or wet weather conditions or between light or dark conditions. These environmental conditions as well as minor incidents are all 'in the mix' of usual operations of motorways as characterised under 'Scope' (refer to Section 3.2) and can only be influenced to a very limited extent. Hence the 'capacity' differences between such conditions are not relevant.

The number of lanes affects per lane 'capacity' as each additional lane induces (disproportionately) more 'friction' between lanes. This is because lane changing activity (disproportionately) increases as additional lanes are added. This phenomenon was considered by determining different per lane capacities for the different cross-section types (i.e. number of lanes) based on measurements at the corresponding locations.

Following the pre-selection, managed motorway 'capacity' values were determined based on a detailed analysis of traffic data collected at twelve Monash and West Gate Freeway measurement sites and two CityLink tunnels over an entire month (October 2015). These sites are identified bottleneck locations. For details regarding the analysed data, refer to Section 3.4.

1-minute speed, flow and density data were aggregated to rolling 1-hour and actual 15-minute averages. It should be noted that density could not be directly measured. Since under uninterrupted conditions flow rate is the product of speed and density, density was calculated based on the following equation:

$$k = q / v$$

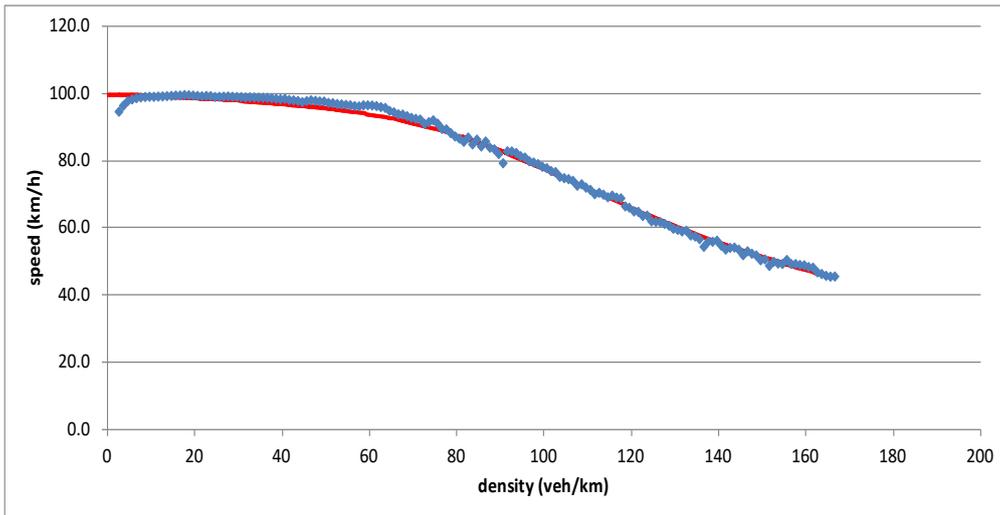
where:

k	=	Density (veh/km)
q	=	Flow rate (veh/h)
v	=	Average vehicle speed (km/h)

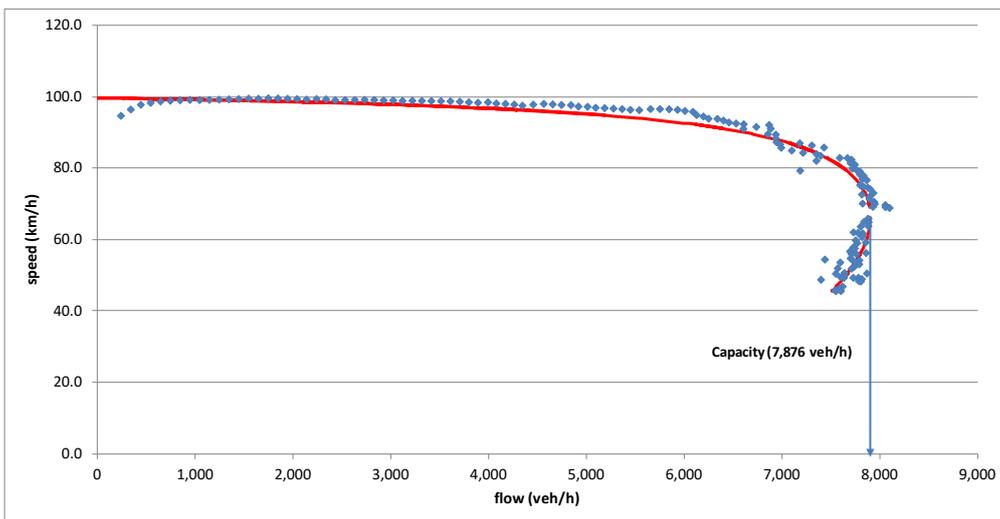
It should be noted that for analysis purposes, the time mean speed (i.e. the average of individual speeds of all vehicles passing a point during a time interval) was used for this calculation as it is very difficult to calculate the space mean speed over a long period of time in many different locations (i.e. the average of individual speeds of all vehicles travelling on a length of roadway) with the currently deployed in-pavement detector technology (could theoretically be done based on individual vehicle speeds). As a rule of thumb, time mean speed is about 2% more than space mean speed.

### 'Capacity' (Approach 1)

'Capacity' is traditionally determined based on the illustration of measured values in speed-density or speed-flow diagrams. As the number of measured (rolling-average) values was very high, these were summarised by calculating average speeds and flows for all 1 hour intervals where density fell within a certain density class with a width of 1 vehicle per kilometre carriageway length (e.g. Density class 1 = 0–1 veh/km, Density class 2 = 1-2 veh/km, etc) - see blue points in the below diagrams (as described in Geistefeldt, 2009). Furthermore, the speed-flow-density relation was abstracted by a model. The only known suitable single regime speed-flow-density relation for motorways and arterials is the so called van Aerde model (van Aerde, 1995). For each cross-section, a corresponding curve was fitted to the point cloud of measured and averaged speed and density values (refer to Figure 3-3 and Step No. 6 below). 'Capacity' is typically defined as the maximum flow rate of the fitted carriageway speed-flow curve (Geistefeldt, 2016). The speed-flow relation and an illustration of 'capacity' value determination can be seen from Figure 3-4.



**Figure 3-3: Speed-density relation for cross-section 14587IB (Monash Freeway - Jacksons to Wellington Road, inbound, 4 lanes) - measured values (1 hour rolling averages aggregated for density classes) and van Aerde curve approximation (elimination of downstream bottleneck influence – hence no very high densities)**



**Figure 3-4: Speed-flow relation for cross-section 14587IB (Monash Freeway - Jacksons to Wellington Road, inbound, 4 lanes) - measured values (1 hour rolling averages aggregated for density classes) and van Aerde curve approximation (elimination of downstream bottleneck influence)**

In summary, the following steps have to be taken for the determination of ‘capacity’ values:

1. Download or otherwise determine speed/volume/occupancy (SVO) data in one minute intervals for one month (spring/autumn)
2. Calculate rolling 1 hour averages for every minute of the month (speed and flow)
3. Calculate rolling 1 hour average density by dividing flow by speed
4. Eliminate all intervals where the rolling 1 hour average speed at the next downstream measurement site was lower than the threshold speed (typically 65 km/h)
5. Calculate average speeds and flows for every density class (0-1 veh/km, 1-2 veh/km, etc) - based on all rolling 1 hour intervals where density fell within a certain class; Figure 3-3 and Figure 3-4 show the actual values as blue dots

6. Fit a van Aerde curve to the corresponding point cloud by determining the optimal parameters C1, C2 and C3 (refer to equation in Section 3.3.2 and red curve in Figure 3-3 and Figure 3-4) –minimisation of the sum of error squares between ‘actual density’ (the independent variable) and ‘van Aerde density’
7. Determine ‘capacity’ as the highest flow rate of the van Aerde curve (refer to Figure 3-4)

**‘Capacity’ (Approach 2, Probability of flow breakdown)**

1. Download or otherwise determine speed/volume/occupancy (SVO) data in one minute intervals for one month (spring/autumn)
2. Calculate 5-minute or 15-minute interval averages for the entire month (speed and flow)
3. Eliminate all intervals where the 5-minute or 15-minute average speed at the next downstream measurement site was lower than the threshold speed (typically 65 km/h)
4. Identify ‘breakdown intervals’ (speed >= threshold speed (typically 65 km/h) in the interval itself; speed < threshold speed in the next interval) and ‘free flow intervals’ (speed >= threshold speed in the interval itself; speed >= threshold speed in the next interval)
5. Identify ‘forced-flow intervals’ that get eliminated from further analysis (speed < threshold speed in the previous interval; speed < threshold speed in the interval itself)
6. Determine the cumulative ‘capacity’ distribution function (also showing the probability of a flow breakdown) by applying the non-parametric Product Limit Method (Brilon et al., 2005):

$$F_c(q) = 1 - \prod_{i: q_i \leq q} \frac{k_i - d_i}{k_i} \quad i \in \{B\}$$

- where:  $F_c(q)$  = Value of the cumulative ‘capacity’ distribution function (CDF) at  $q$   
 $k_i$  = Number of intervals with a flow rate of  $q \geq q_i$ ,  
 $d_i$  = Number of breakdowns at a flow rate of  $q_i$  (usually 1), and  
 $\{B\}$  = Set of breakdown intervals (Interval B).

The product in the above equation is calculated over all observed 5- or 15-minute intervals  $i$  with flow rates  $q_i \leq q$ , each of which was a ‘breakdown interval’. Usually, each observed ‘breakdown interval’ is used as one  $q_i$  value, so that  $d_i$  is always equal to 1. The distribution function will reach a value of 1 only if the maximum observed flow rate is a B value (i.e. a breakdown followed). Otherwise, the distribution function terminates at a value of  $F_c(q) < 1$ . In this case, the method does not allow complete estimation of the function  $F_c(q)$ .

A parametric estimation based on the minimisation of the sum of error squares principle or similar can be applied to receive a complete distribution function. The error is defined as the difference between the nonparametric and the parametric value for  $F_c(q)$  for a particular flow rate  $q_i$ . According to (Brilon et al., 2005), a comparison between different mathematical types of functions revealed the best results for the Weibull distribution. He states that empirical analysis of traffic flow patterns over several months and at many sites clearly showed that ‘capacity’ as defined in this context is Weibull-distributed.

A Weibull curve was first fitted to the full range of values of the CDF/probabilities of flow breakdown (refer to Section 3.5.4). Secondly, in order to get an optimised fit in the lower value range, this was also done for a value of the CDF/probability of flow breakdown of up to 15%. It turned out that the 1% breakdown probability in the first case occurred at 92% of van Aerde ‘Capacity’ (Approach 1) and in the second case at 91% (i.e. the difference is very small). Given that there are usually only few flow breakdowns in the lower value range, it is considered that the level of randomness of the results is higher when only focussing on the lower value range and that analysing the full value range gives more robust results (in particular because this approach is consistent with research including (Brilon et al., 2005)). The first approach is therefore seen as more suitable for the task of providing Maximum

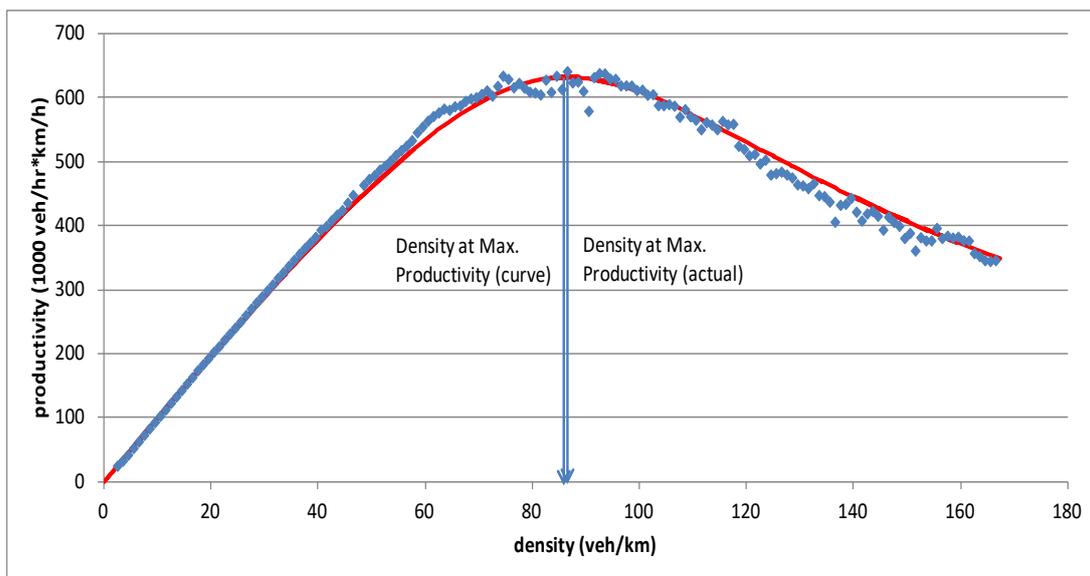
Sustainable Flow Rates for planning and design. For operational optimisation, the second approach is seen as equally important.

For the determination of the Maximum Sustainable Flow Rates (MSFR) in Section 2.2, flow rates at 1% probability of flow breakdown were normalised to consider gradients  $\geq 2\%$  and HGV percentages other than 15% (based on the factors included in Section 3.7 and as applied in Table 3-1). MSFR for surface 3-lane cross-sections were determined by interpolating the values for 2-lane and 4-lane cross-sections. This was done for the following reasons:

- The selected 3-lane measurement sites have limited validity as bottlenecks, and
- Compared to the other cross-sections, the fitting of the Weibull curve to measured points is more difficult here (refer to Diagrams for 3 lane cross-sections in Section 3.5.4).

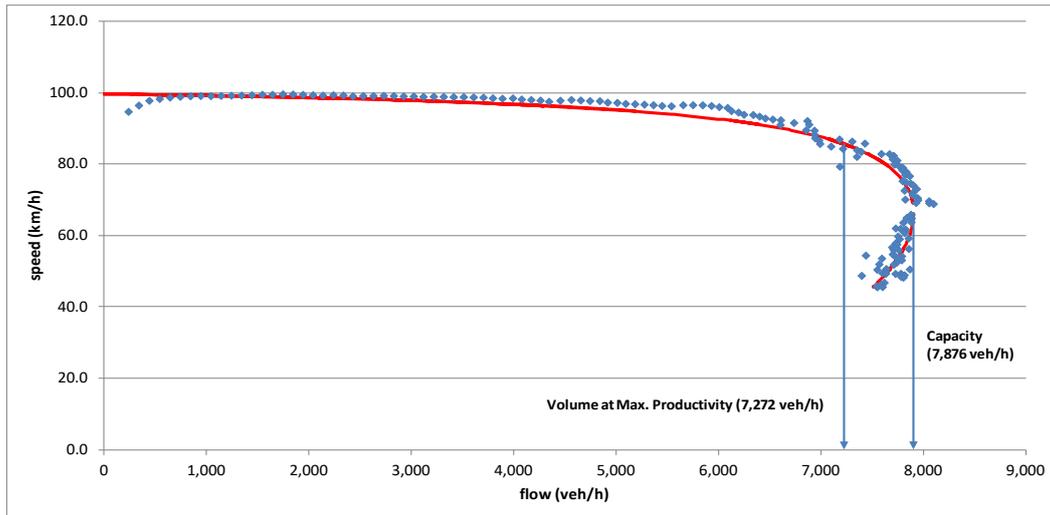
**Productivity**

Similar to ‘capacity’ determination (Approach 1), as the number of measured (rolling-average) values was very high, these were summarised by calculating the average productivity for all 1 hour intervals where density fell within a certain density class with a width of 1 vehicle per kilometre carriageway length (e.g. Density class 1 = 0–1 veh/km, Density class 2 = 1-2 veh/km, etc) - see blue points in Figure 3-5. Subsequently, for each cross-section a new van Aerde curve (independent of that used for the determination of ‘capacity’ values) was fitted to the point cloud of measured and averaged productivity values.



**Figure 3-5: Carriageway productivity-density relation for cross-section 145871B (1 hour rolling averages aggregated for density classes)**

For illustration, Figure 3-6 shows the flow rate at maximum productivity compared to ‘capacity’.



**Figure 3-6: ‘Capacity’ and volume at maximum productivity for cross-section 145871B (1 hour rolling averages aggregated for density classes)**

The measurement results presented in Table 3-1 were normalised to represent ‘standard’ conditions which were defined as a gradient smaller or equal to 2% and a Heavy Goods Vehicle (HGV) percentage of 15%.

No normalisation was done for other ‘non-standard’ static conditions (refer to Table 1-2):

- Lane and shoulder widths – Lane widths at all measurement cross-sections  $\geq 3.25$  metres; no shoulder at Cross sections 14779 IB, 14427WB, and 14428 EB (impact of the absence of shoulders on ‘capacity’ assumed to be minor, refer to (Henkens and Heikoop, 2015), Section 4.1.3);
- Visibility conditions;
- Curvature - narrow curve at Cross-section 14779 IB (regarding the impact of tight curves on ‘capacity’ refer to Section 3.9);
- Sags and crests (particularly relevant to tunnels) – separate values determined for tunnels;
- Location – covered by the scope of this Guide (refer to Section 3.2).

No attempt has been made to standardise measurement results due to dynamic factors as listed in Table 1-3.

## 3.4 Data and Site Observations

This section outlines objectives for the collation of data used for this Guide. A qualification of the data used and observations from individual measurement sites are also provided.

### 3.4.1 Data collection principles

It was intended to select cross-sections for the collation of data to estimate ‘capacity’ values and Maximum Sustainable Flow Rates that represent the following conditions:

- Represent all relevant conditions (2-lane, 3-lane, 4-lane, 5-lane cross-sections; different gradients and HGV percentage categories; ‘normal sections’ or bridges/tunnels).
- Collect data in all weather and light conditions (i.e. dry and wet, light and dark).
- Exclude the majority of incident influence.
- It was not considered possible to estimate capacities for all combinations of these factors; e.g. the ‘German HCM’ sets ‘capacity’ values for more than 200 different combinations but is based on only around 50 measurement sites.

- Where there were gaps in the VicRoads data, appropriate information in other highway capacity manuals was used to supplement information, for example data published in the ‘German HCM’ or the ‘Dutch HCM’ was used to interpolate and extrapolate own findings (compare to Section 3.7).

### 3.4.2 Qualification of the data used

All data used was from the M1 Corridor (Monash and West Gate Freeways). This is because it was the only managed motorway corridor where high quality data was available for the analysis at the time of the analysis.

During the analysis period, not all the requirements for a managed motorway as outlined in Section 3.2.2 had been fully met. For example, the access from Eastlink to the Monash Freeway in the inbound direction was uncontrolled. Hence, the affected motorway sections have to be considered as ‘partially managed’. It is expected that ‘capacity’ values at selected cross-sections will go up after implementation of the corresponding improvements (e.g. at Detector 14587IB). Realistically, partially managed motorways are the norm since it may never be possible to fully meet all requirements.

## 3.5 Measurement results

### 3.5.1 ‘Capacity’

Table 3-1 shows ‘capacity’ values as determined at a number of locations on Melbourne’s motorway network. The corresponding methodology is explained in Sections 3.3.2 (Approach 1) and 3.3.3.

Detector	Section from	Section to	Capacity	Number of lanes	Capacity per lane	Gradient	HGV percentage	Normalised Capacity per lane 5)
			veh/h		veh/h/lane	%	%	veh/h/lane
14456 IB	Ernst-Wanke/Tinks	Belgrave-Hallam	3,963	2	1,982	2.5	11	1,954
14456 OB	Belgrave-Hallam	Ernst-Wanke/Tinks	4,111	2	2,056	<=2	11	1,976
14542 IB	Warrigal	High	7,280	4	1,820	<=2	13	1,784
14547 OB 1)	High	Warrigal	5,176	3	1,725	<=2	13	1,692
14547 OB 2)	High	Warrigal	6,652	4	1,663	<=2	13	1,630
14571 IB 1)	Huntingdale	Warrigal	6,140	3	2,047	<=2	13	2,007
14571 IB 2)	Huntingdale	Warrigal	7,342	4	1,836	<=2	13	1,800
14573 OB	Huntingdale	Forster	7,938	4	1,985	<=2	11	1,908
14779 IB	Toorak	Yarra Blvd	7,334	4	1,834	<=2	15	1,834
14587 IB	Jacksons	Wellington	7,876	4	1,969	<=2	11	1,893
14428 EB 3)	Williamstown	Todd	8,417	5	1,683	3.5	17	1,757
14427 WB 3)	Todd	Williamstown	8,451	5	1,690	3.9	17	1,803
14762 IB 4)	Punt Road	Kings Way	5,191	3	1,730	<=2	18	1,773
14769 OB 4)	Kings Way	Burnley Street	4,444	3	1,481	5.2	18	1,829

- 1) Only considering the three lanes continuing over Warrigal Road Bridge
- 2) Considering all four lanes at the cross-section
- 3) Different ‘non-standard’ conditions including gradient, lane-changing restrictions (assumed to increase ‘capacity’ by 5%), higher HGV percentage
- 4) Domain and Burnley Tunnels
- 5) Considering gradient, HGV-percentage and lane changing conditions at measurement sites (normalised to  $s \leq 2\%$ , HGV = 15% based on the factors shown in Section 3.7)

**Table 3-1: ‘Capacity’ values**

### 3.5.2 Maximum productivity

Productivity is defined as the product of traffic flow and traffic speed and it’s achieved if both speed and flow are maintained near maximum values, i.e. near free-flow speed and capacity flow. For each managed motorway cross-section that was used for ‘capacity’ analysis, the traffic flow was determined for which the arithmetic product of these two Key Performance Indicators (KPI’s) can be expected to reach its maximum (refer to Section 3.3.3).

## Part 3: VicRoads Motorway Capacity Guide

Detector	Capacity	Number of lanes	Volume at Max Productivity (actual)		Volume at Max Productivity (curve)	
	veh/h		veh/h	% of vA Capacity	veh/h	% of vA Capacity
14456 IB	3,963	2	3,814	96	3,814	96
14456 OB	4,111	2	3,055	74	3,511	85
14542 IB	7,280	4	7,015	96	6,949	95
14547 OB 1)	5,176	3	4,743	92	4,860	94
14547 OB 2)	6,652	4	6,099	92	6,283	94
14571 IB 1)	6,140	3	5,894	96	5,959	97
14571 IB 2)	7,342	4	5,109	70	6,640	90
14573 OB	7,938	4	7,921	100	7,748	98
14779 IB	7,334	4	7,100	97	7,100	97
14587 IB	7,876	4	7,272	92	7,346	93
14428 EB 3)	8,417	5	7,950	94	7,919	94
14427 WB 3)	8,451	5	8,044	95	8,241	98
14762 IB	5,191	3	4,583	88	4,754	92
14769 OB	4,444	3	4,444	100	3,913	88
			<b>Average:</b>	<b>91.6</b>	<b>Average:</b>	<b>93.7</b>
			<b>Standard Deviation:</b>	<b>9.0</b>	<b>Standard Deviation:</b>	<b>3.7</b>

- 1) Only considering the three lanes continuing over Warrigal Road Bridge
- 2) Considering all four lanes at the cross-section
- 3) Different 'non-standard' conditions including gradient, lane-changing restrictions (assumed to increase 'capacity' by 5%), higher HGV percentage

**Table 3-2: 'Capacity' values compared to flows at maximum productivity**

### 3.5.3 Probability of flow breakdown

Traffic flow breakdown is defined as the transition from 'free flow to 'forced flow' (refer to Section 1.2) which coincides with a significant loss in productivity (i.e. traffic speed and traffic flow).

Table 3-3 lists flow rates for the different managed motorway cross-sections used for 'capacity' analysis that correspond to certain traffic flow breakdown probabilities (15 minute intervals) and links them back to 'capacity' values. The corresponding methodology is explained in Sections 3.3.2 (Approach 2, Variant b)) and 3.3.3.

Detector	1% Breakdown probability		2% Breakdown probability		5% Breakdown probability		50% Breakdown probability	
	veh/h	% of C	veh/h	% of C	veh/h	% of C	veh/h	% of C
14456 IB	3,803	96	3,906	99	4,049	102	4,475	113
14456 OB	3,638	88	3,783	92	3,986	97	4,614	112
14542 IB	6,829	94	7,014	96	7,270	100	8,035	110
14547 OB	4,876	94	5,021	97	5,222	101	5,826	113
14547 OB	6,174	93	6,330	95	6,545	98	7,183	108
14571 IB	5,408	88	5,626	92	5,929	97	6,867	112
14571 IB	6,818	93	6,986	95	7,217	98	7,903	108
14573 OB	7,330	92	7,568	95	7,897	99	8,897	112
14779 IB	6,940	95	7,074	96	7,258	99	7,796	106
14587 IB	7,528	96	7,742	98	8,036	102	8,918	113
14428 EB	7,807	93	8,027	95	8,330	99	9,239	110
14427 WB	7,471	88	7,716	91	8,054	95	9,080	107
14762 IB	4,784	92	4,931	95	5,135	99	5,749	111
14769 OB	4,081	92	4,313	97	4,643	104	5,708	128
Average percentage of Capacity		92		95		99		112
Minimum		88		91		95		106
Maximum		96		99		104		128
Expected breakdown probability		11.4		21.5		46.0		100.0

**Table 3-3: Flow rates for different traffic flow breakdown probabilities (not normalised)**

### 3.5.4 Qualitative description of measurement sites

All curves shown below are based on data that was largely unaffected by congestion spilling back from downstream sections. This was achieved by eliminating those 1 hour or 15 minute intervals where the speed at the next downstream cross-section was lower than a threshold speed (compare to Section 3.3.3).

The first diagram shows the hourly speed-density relationship according to van Aerde (Section 3.3.2, Approach 1). As the number of measured values was very high, the blue dots shown are average speeds for all rolling 1 hour intervals where density fell within a certain density class with a width of 1 vehicle per kilometre carriageway length (refer to Section 3.3.3).

The second diagram shows the hourly speed-flow relationship and 'capacity' according to van Aerde (Section 3.3.2, Approach 1). Again, as the number of measured values was very high, the blue dots shown are average speeds and flows for all rolling 1 hour intervals where density fell within a certain density class with a width of 1 vehicle per kilometre carriageway length.

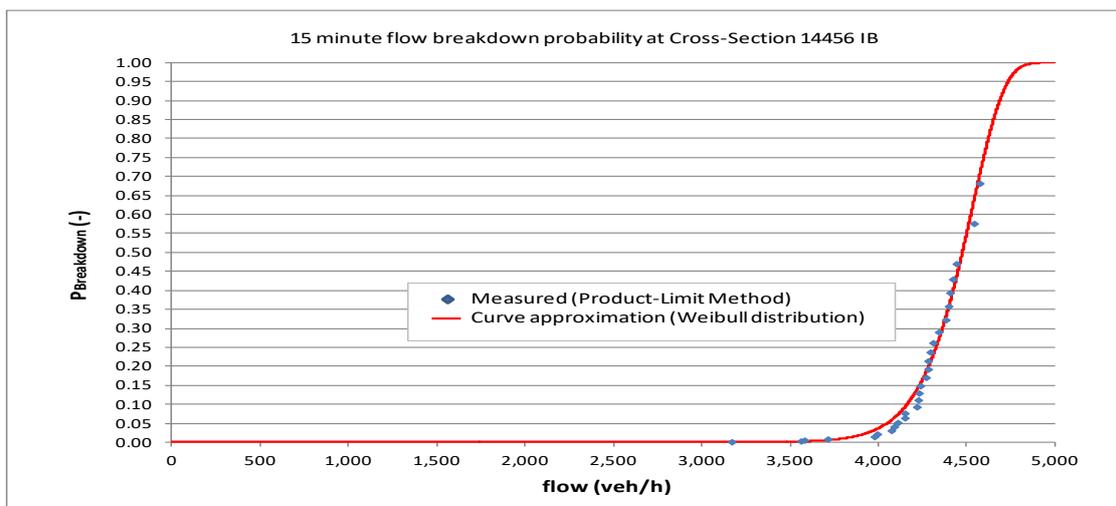
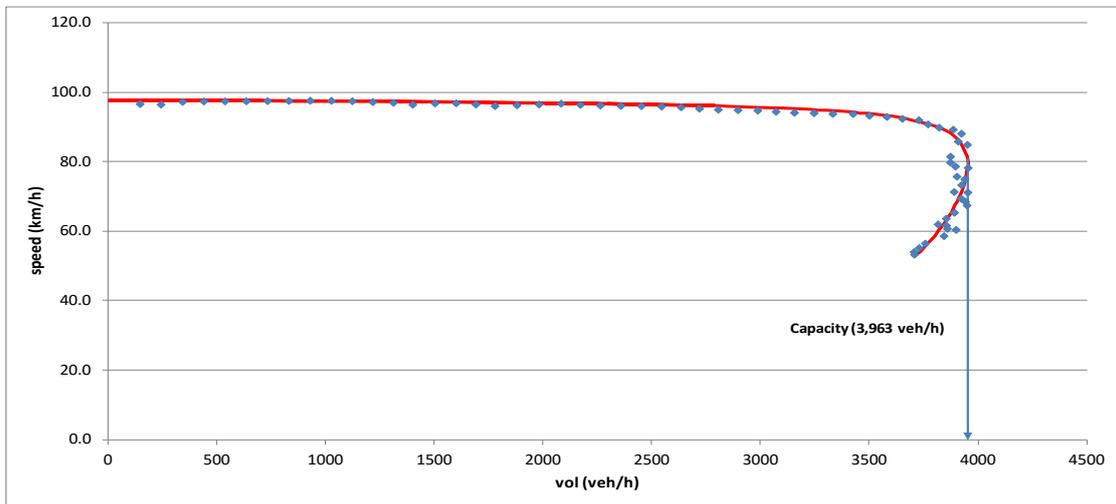
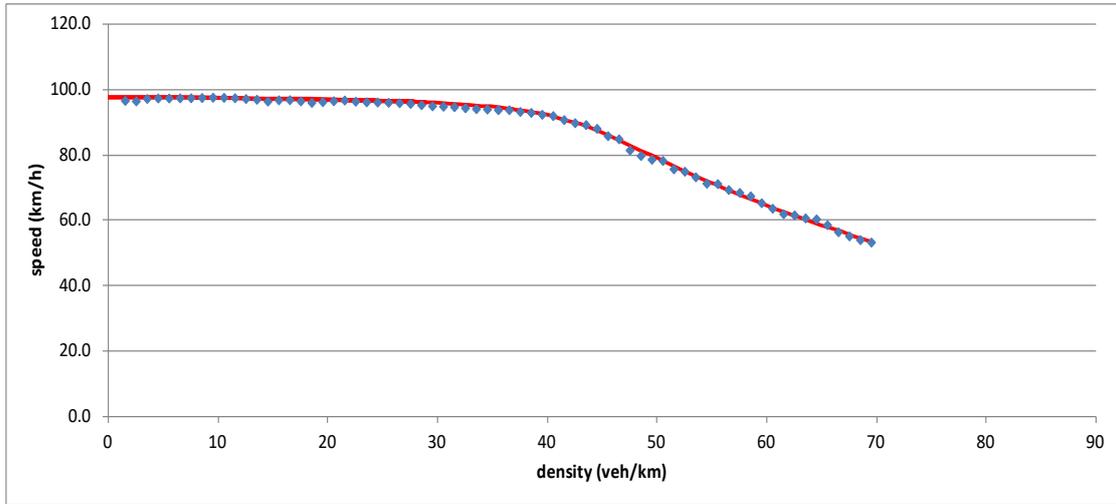
The third diagram shows the 15 minute interval cumulative 'capacity' distribution function or probability of flow breakdown as a function of carriageway flow rate (Section 3.3.2, Approach 2, Variant b)). These diagrams show the probability of an abrupt speed reduction/transition into the 'forced flow' area of the fundamental diagram for an average flow rate  $x$  maintained over a 15 minute period (as per the x-axis); this probability is determined by the corresponding value on the y-axis. In contrast to (Brilon et al., 2005) who used data aggregated over 5 minute intervals the analysis was based on 15 minute intervals.

This was done for the following reasons:

- The relationship between measured flow rate and probability of flow breakdown is better than for 5 minute intervals (i.e. more of the highest flow rate values led to a flow breakdown and hence the curves are better defined in the upper probability range).
- The potential aggregation to 1 hour 'capacity' values is less problematic than for 5 minute intervals.
- 15 minute intervals were also available for the before M1 Upgrade situation which allowed for an easy comparison between the before (unmanaged) and after (managed) situations (compare to Section 3.8).

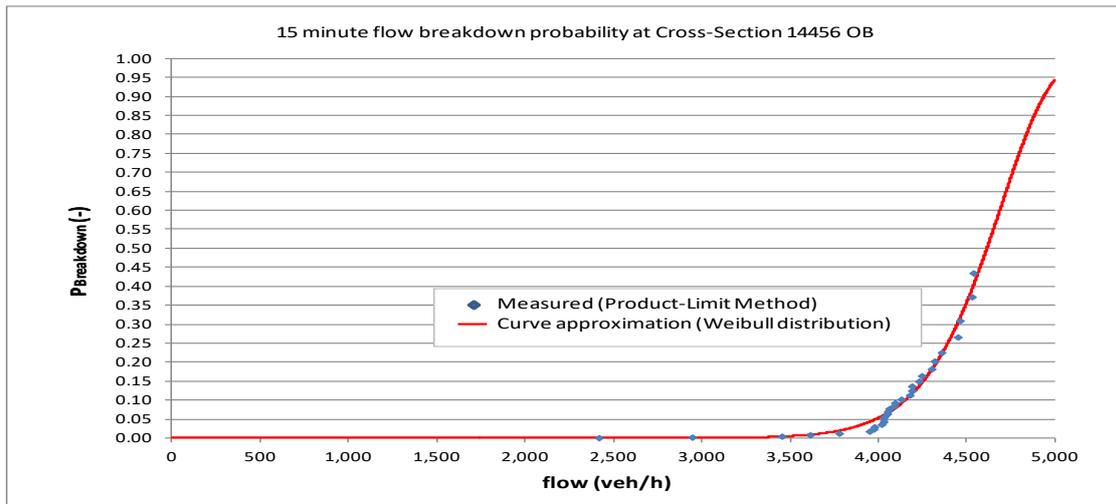
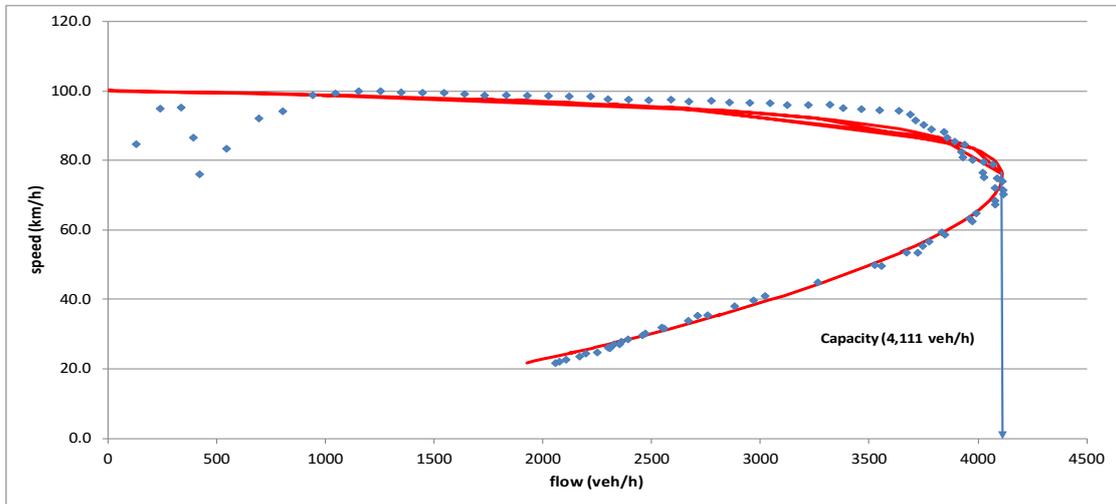
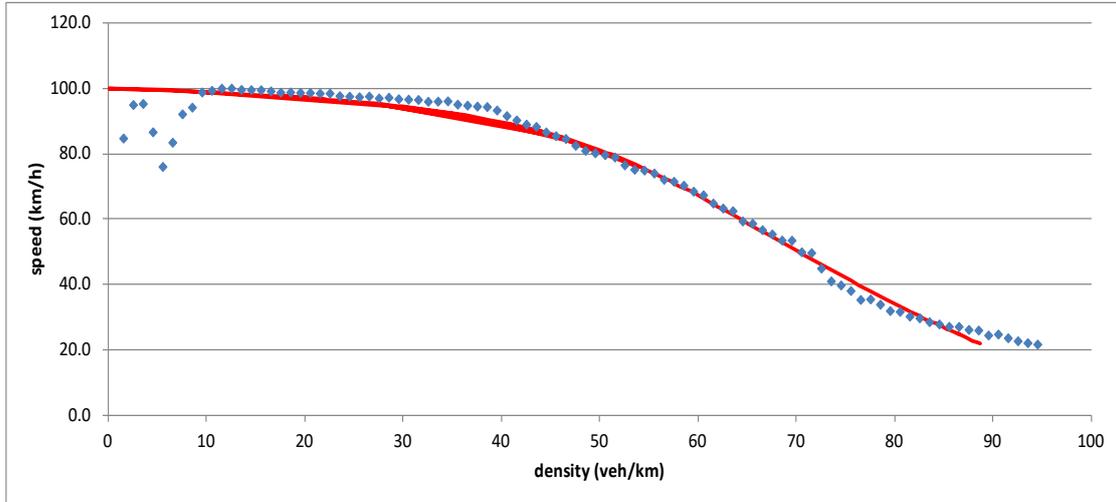
14456 IB Ernst-Wanke/Tinks to Belgrave-Hallam

'Maybe some friction as only 800 metres between on-/off-ramps; no curves' (s=2.5% over <600 m length)



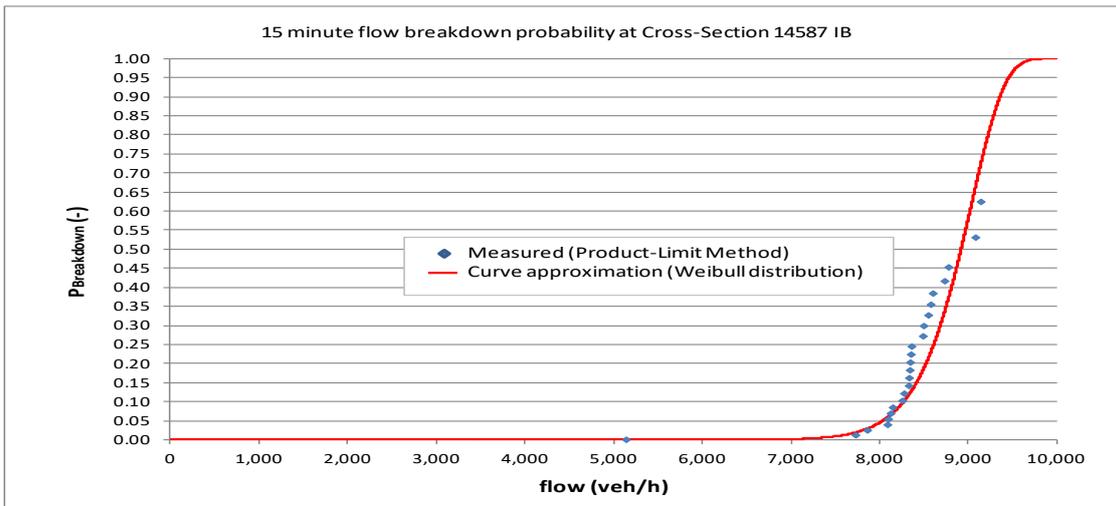
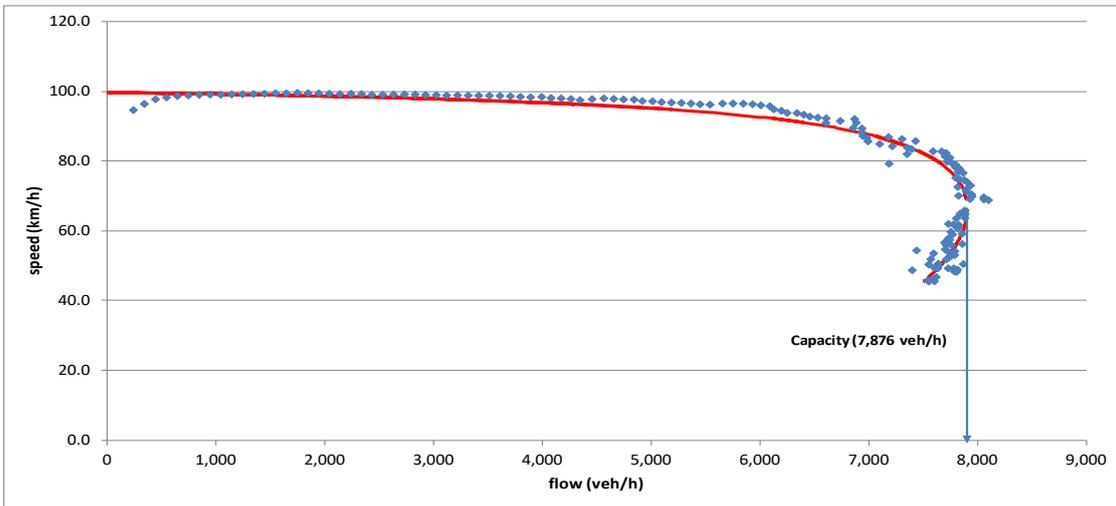
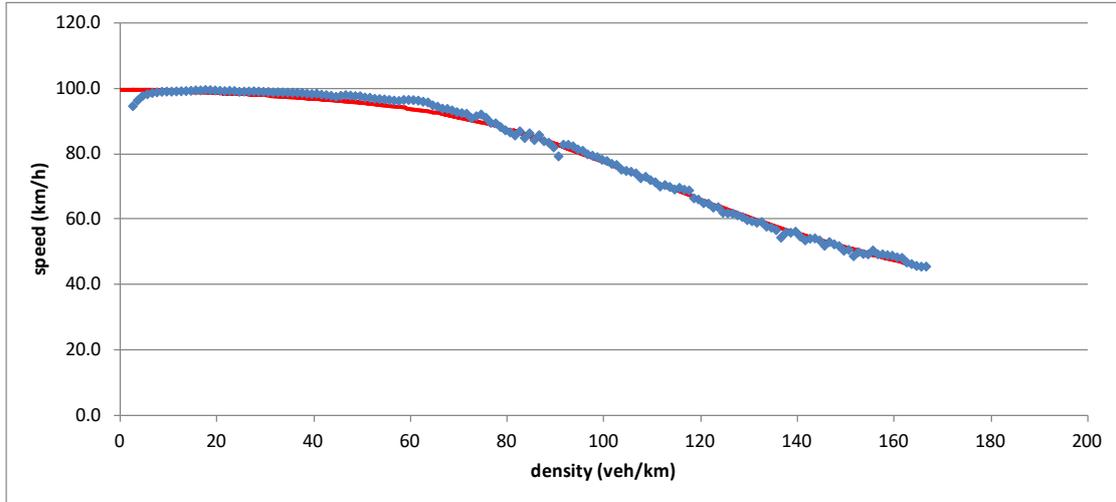
14456 OB Belgrave-Hallam to Ernst-Wanke/Tinks

As above ( $s \leq 2.0\%$ )



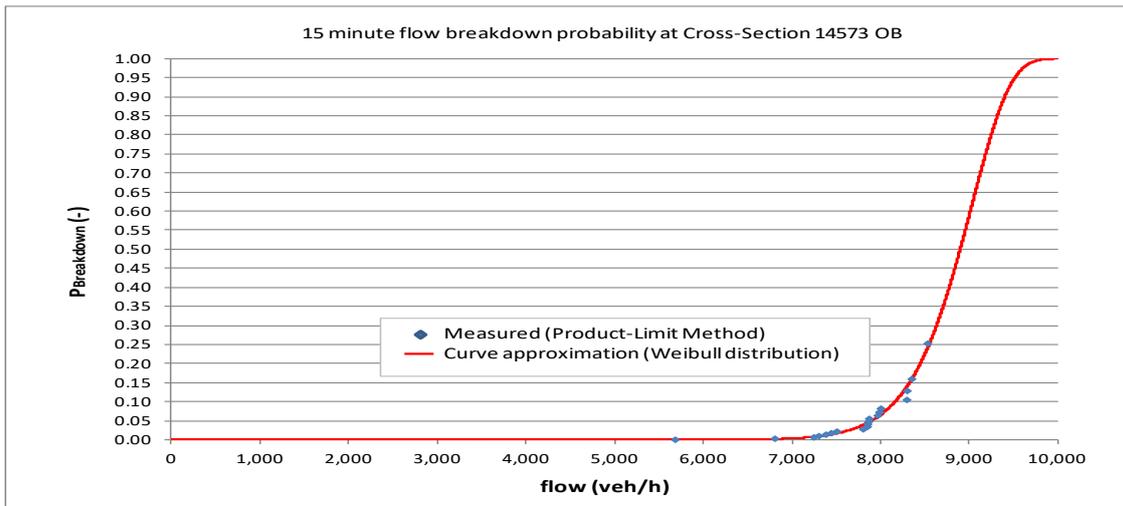
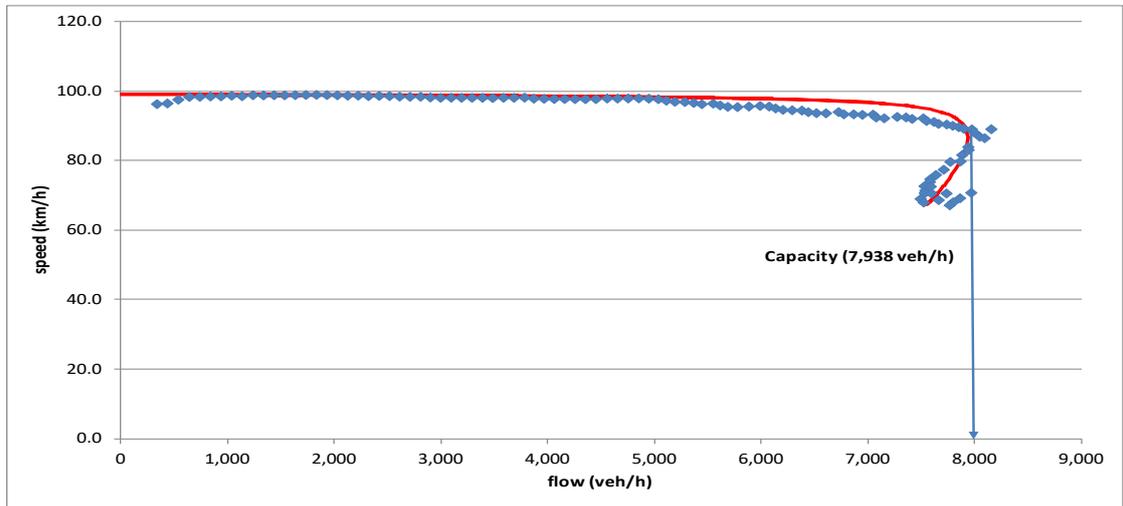
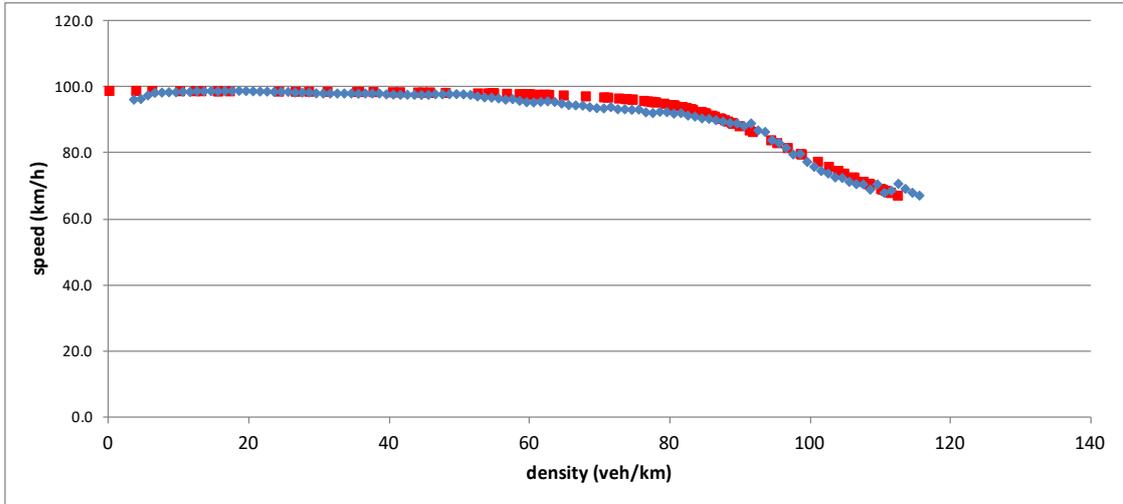
14587 IB Jacksons to Wellington

'EastLink on-ramp merge (lane reduction) immediately before + Jacksons Road on-ramp' ( $s \leq 2.0\%$ )



14573 OB Huntingdale to Forster

'East of Huntingdale and close to creek; no significant curves' ( $s \leq 2.0\%$ )



14571 IB Huntingdale to Warrigal

'East of Warrigal; fourth lane exiting and three lanes continuing (similar to 14547 OB but better geometrically)' ( $s \leq 2.0\%$ )

Diagrams for 4 lanes

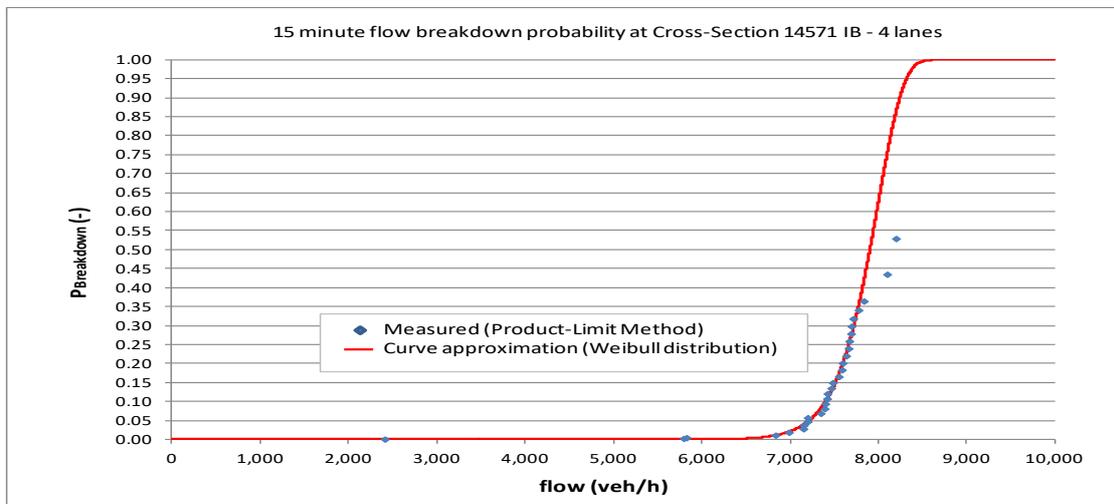
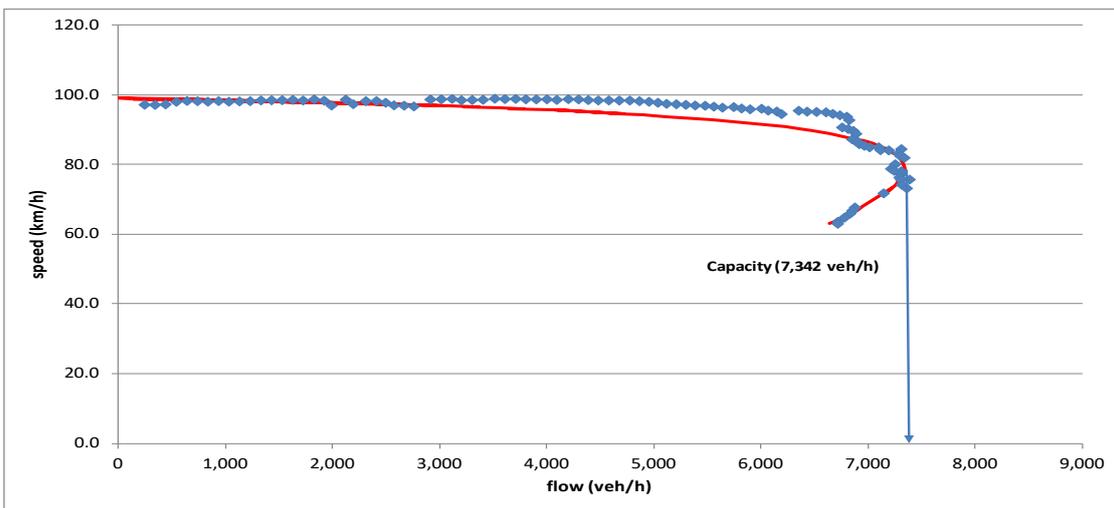
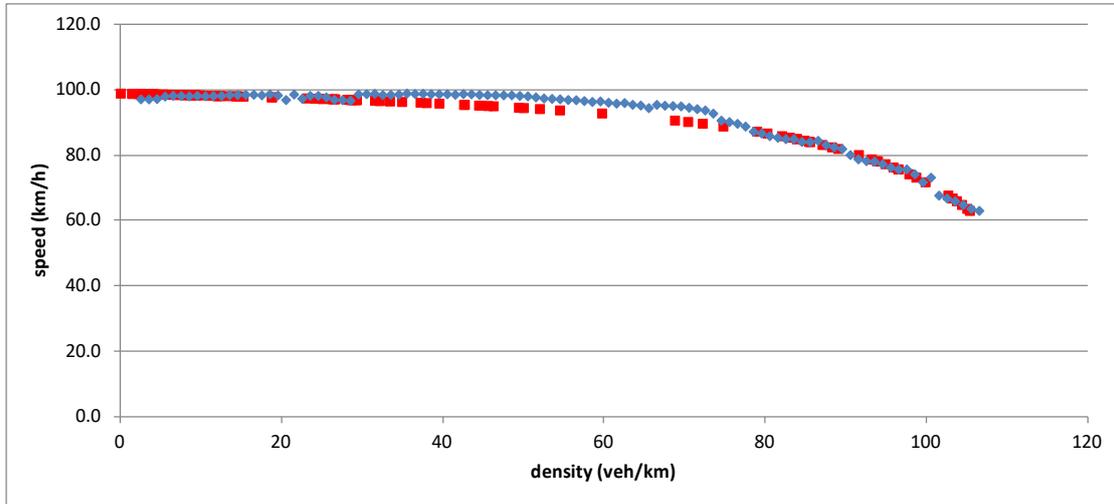
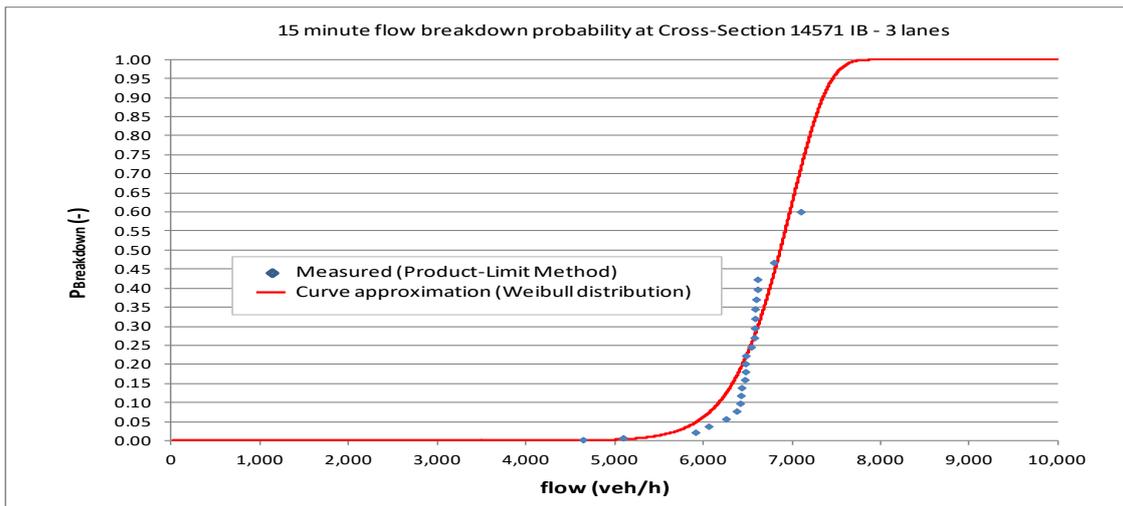
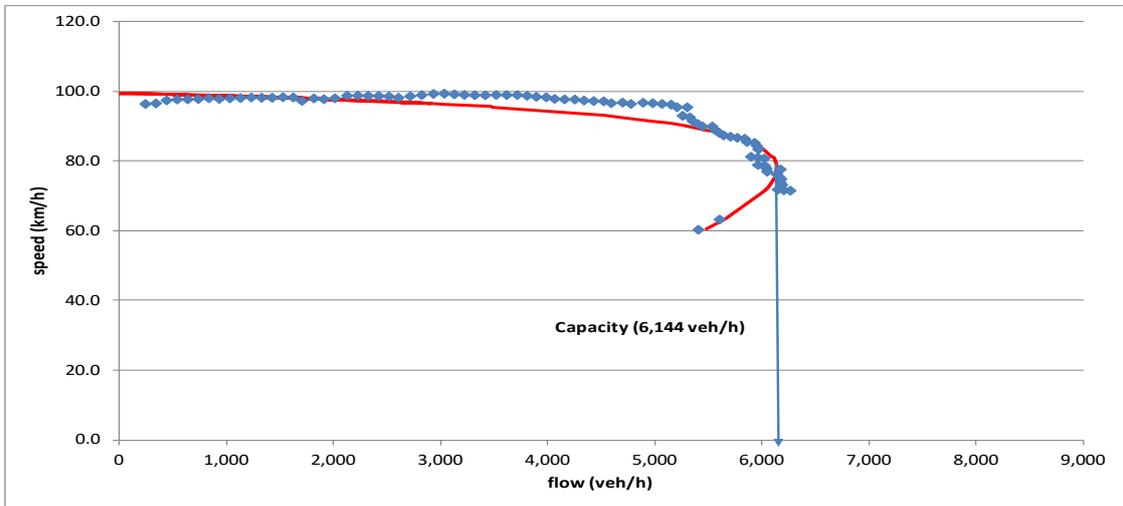
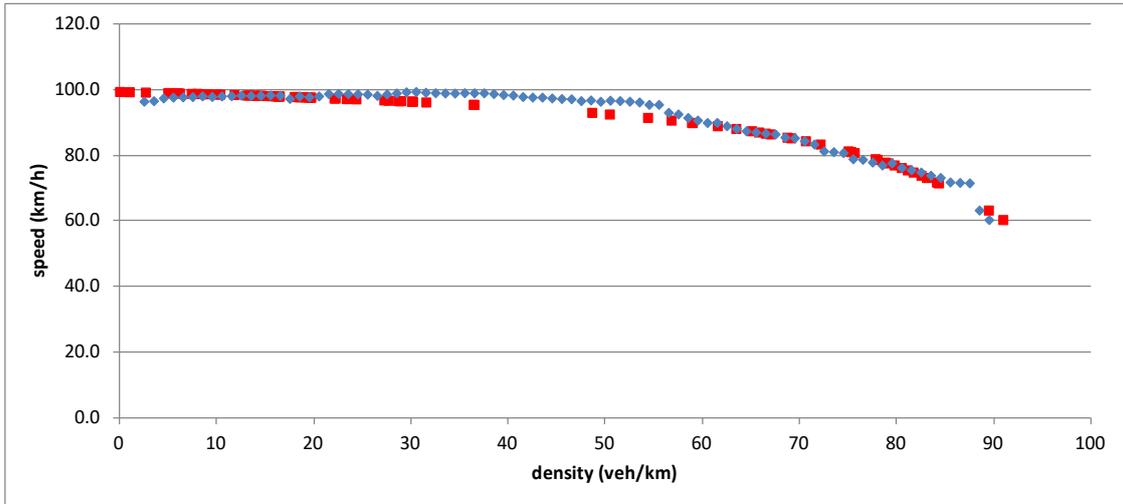
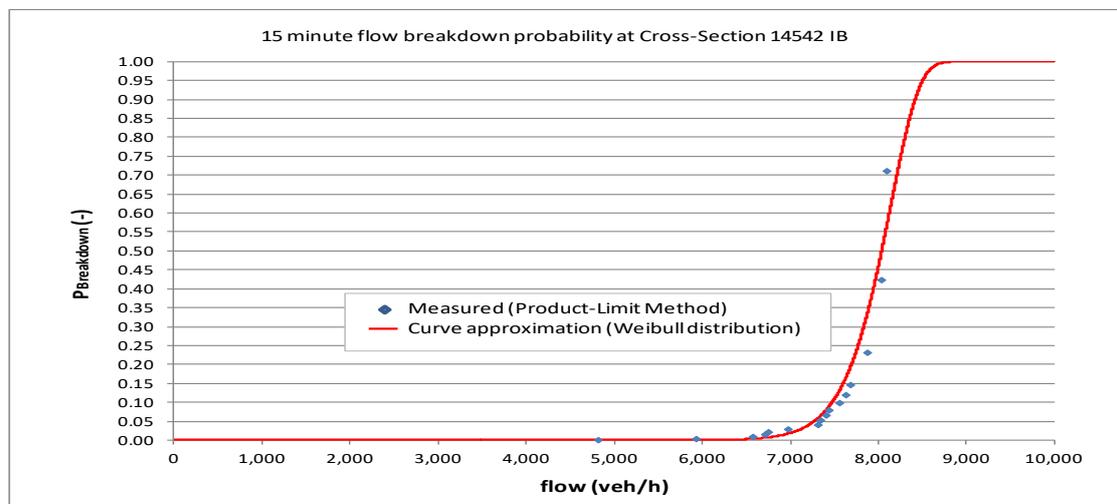
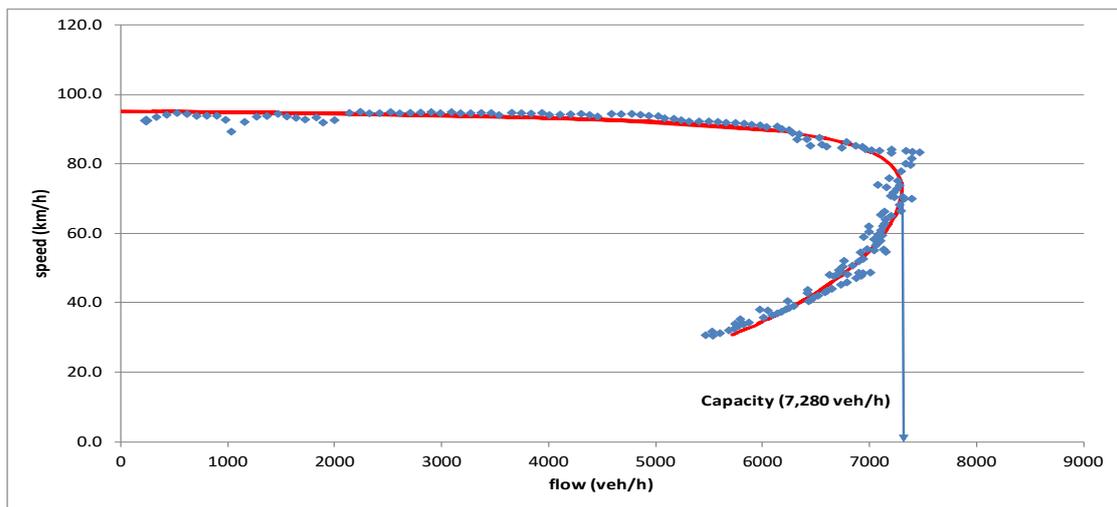
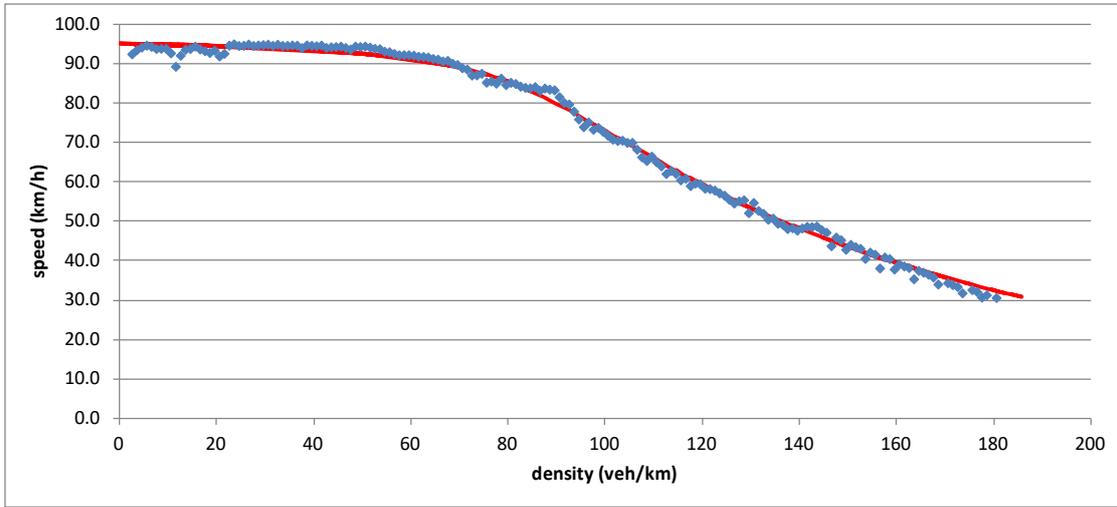


Diagram for 3 lanes, refer to comment in Section 3.3.3, bottom of sub-section 'Capacity' (Approach 2)



14542 IB Warrigal to High

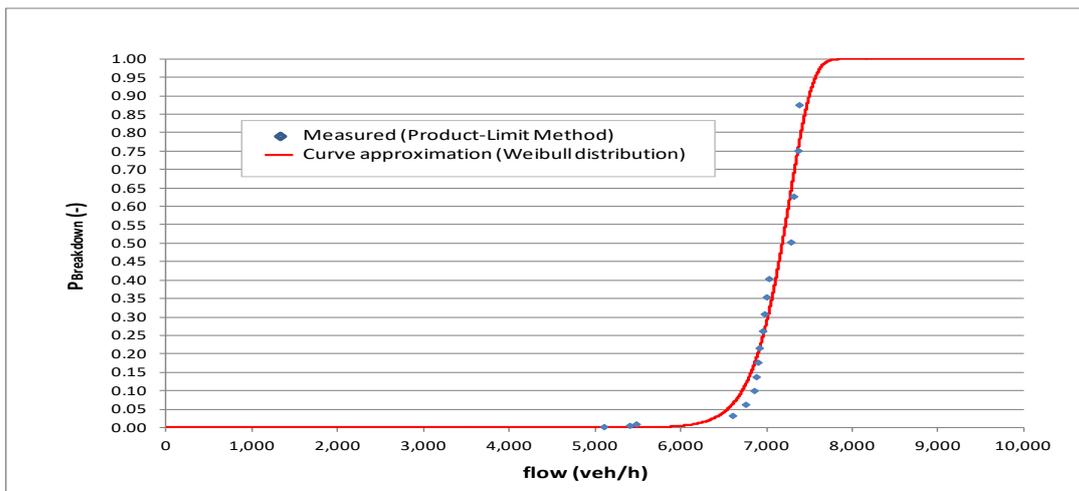
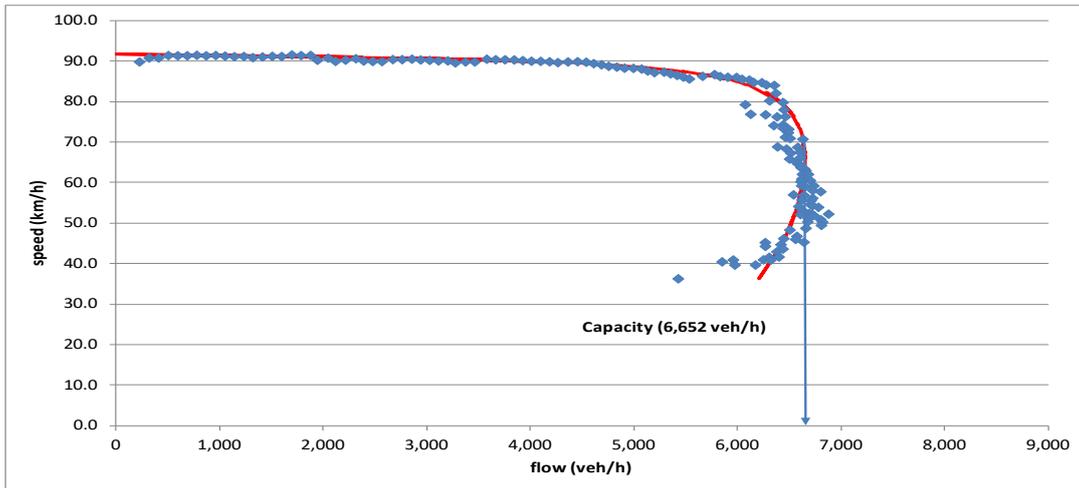
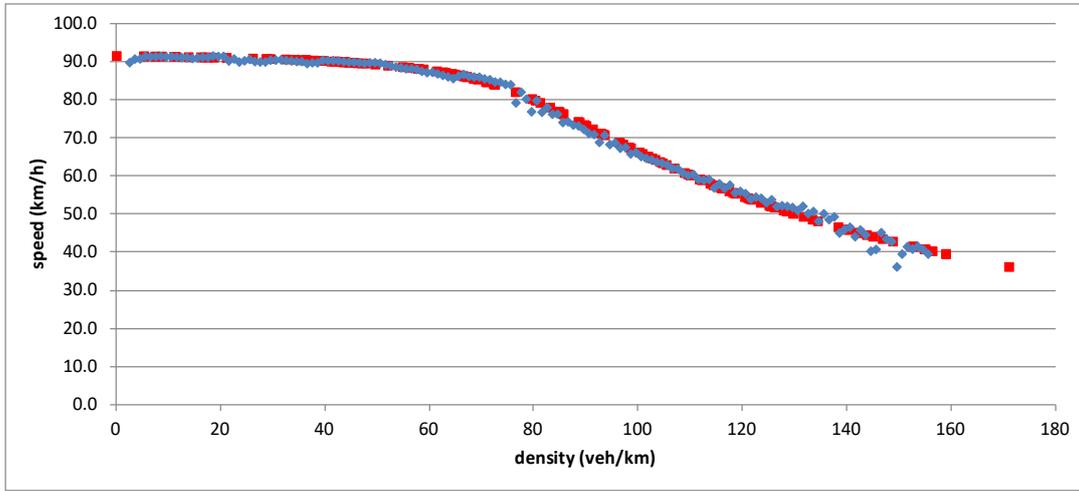
'Just east of High Street (additional lane from here); no shoulders and presence of sidewalls give a tunnel impression + lots of lane changes because of fifth lane downstream; no significant curves or grades' ( $s \leq 2.0\%$ )



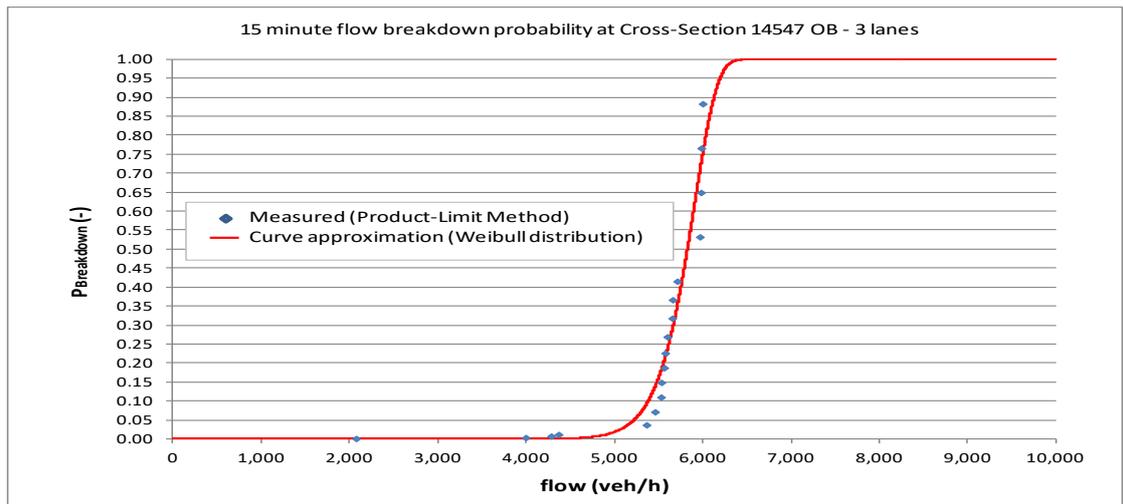
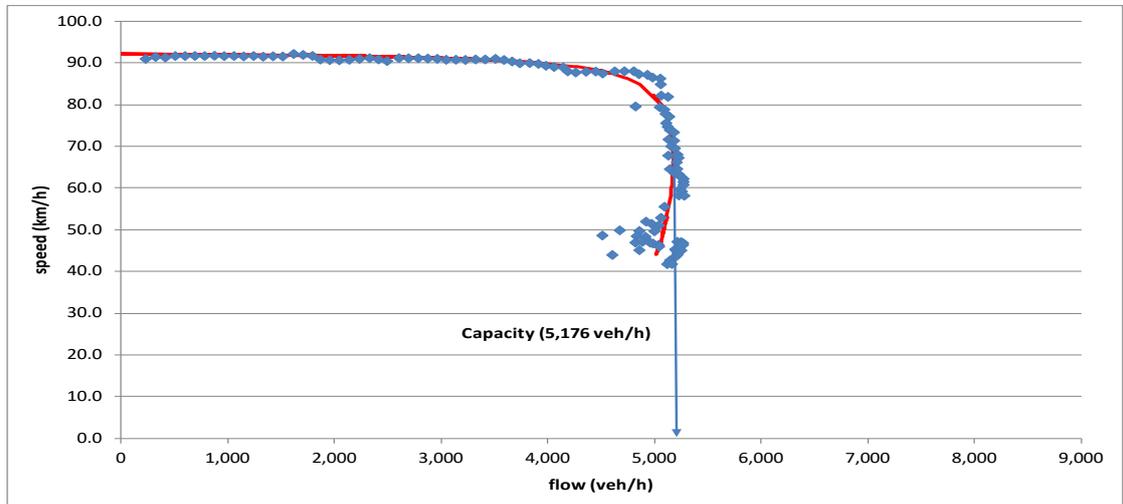
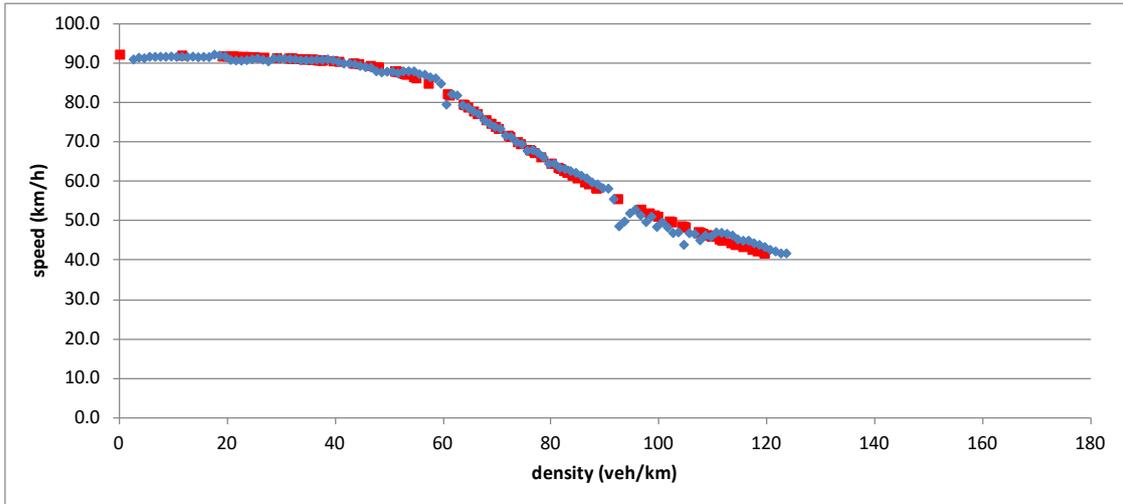
14547 OB High to Warrigal

'Fourth lane exiting and three lanes continuing over the bridge (analysis repeated using measurement sites on Warrigal Bridge – compare to comments under Table 3-1)' ( $s \leq 2.0\%$ )

Diagrams for 4 lanes

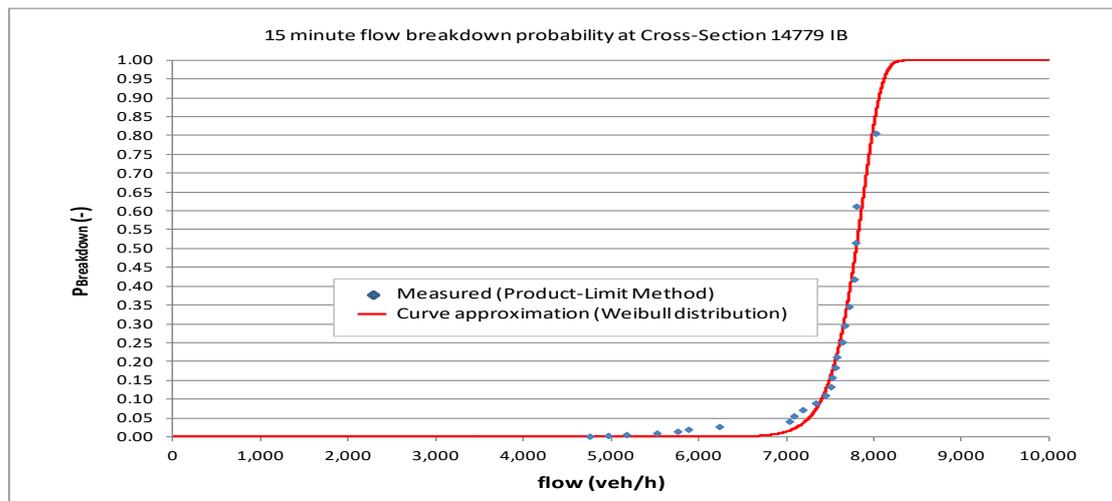
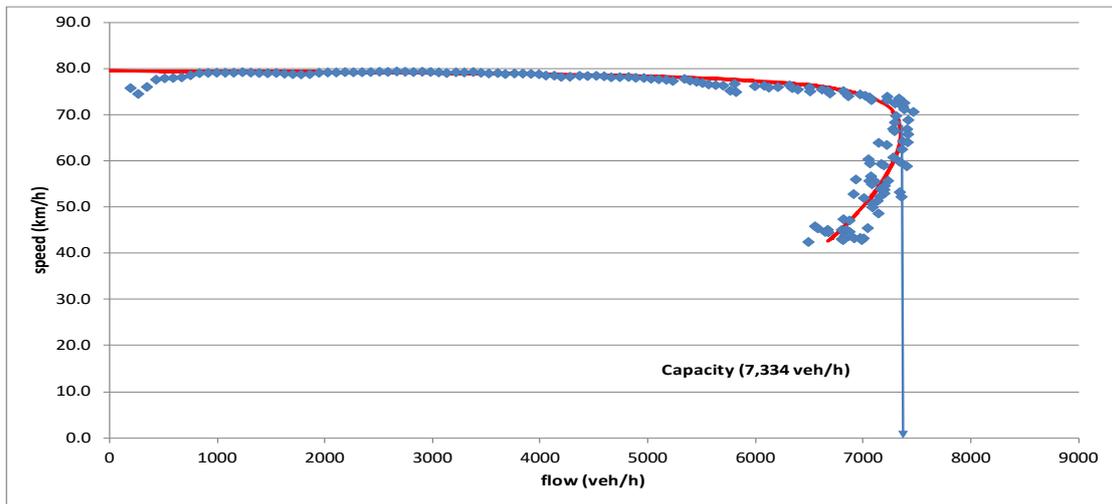
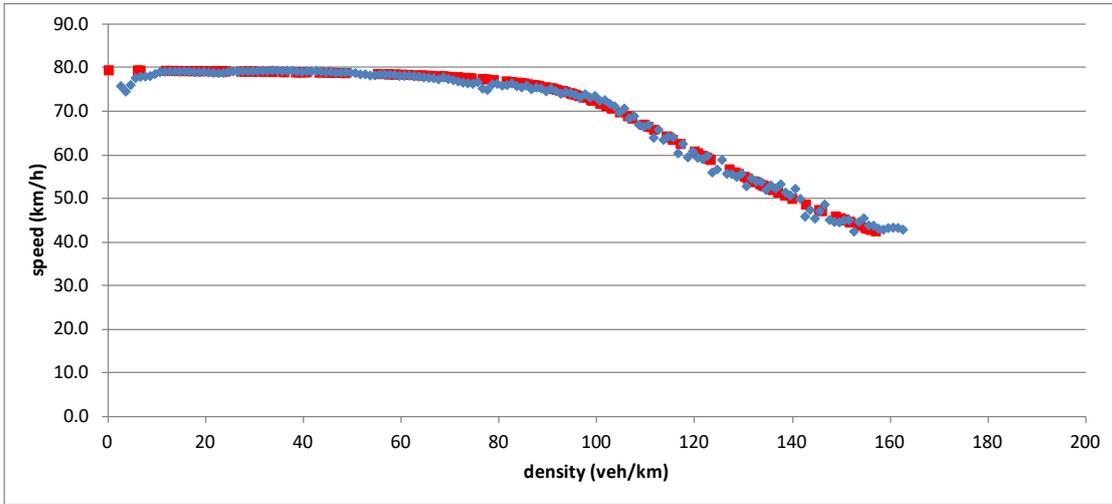


Diagrams for 3 lane refer to comment in Section 3.3.3, bottom of sub-section 'Capacity' (Approach 2)



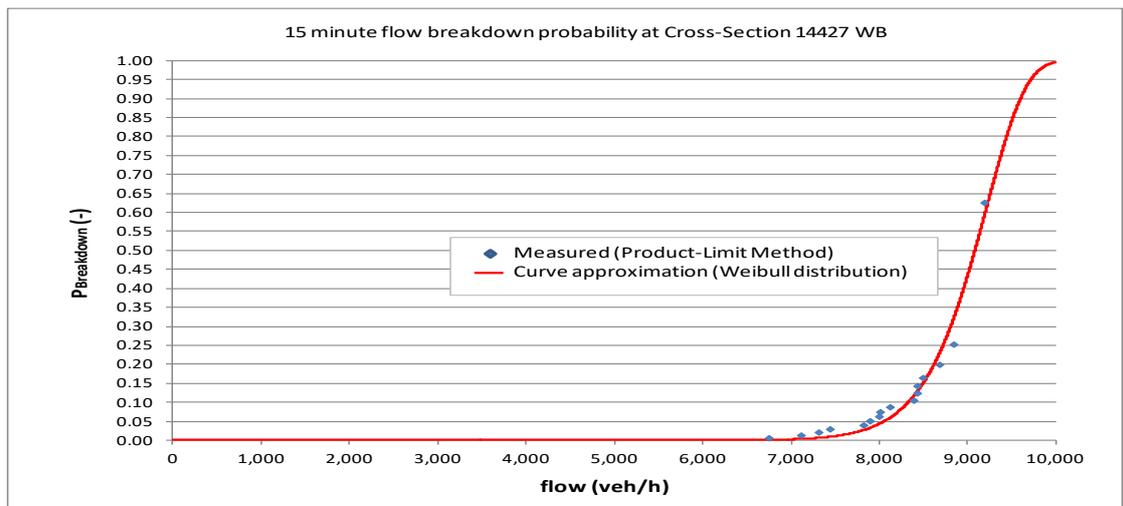
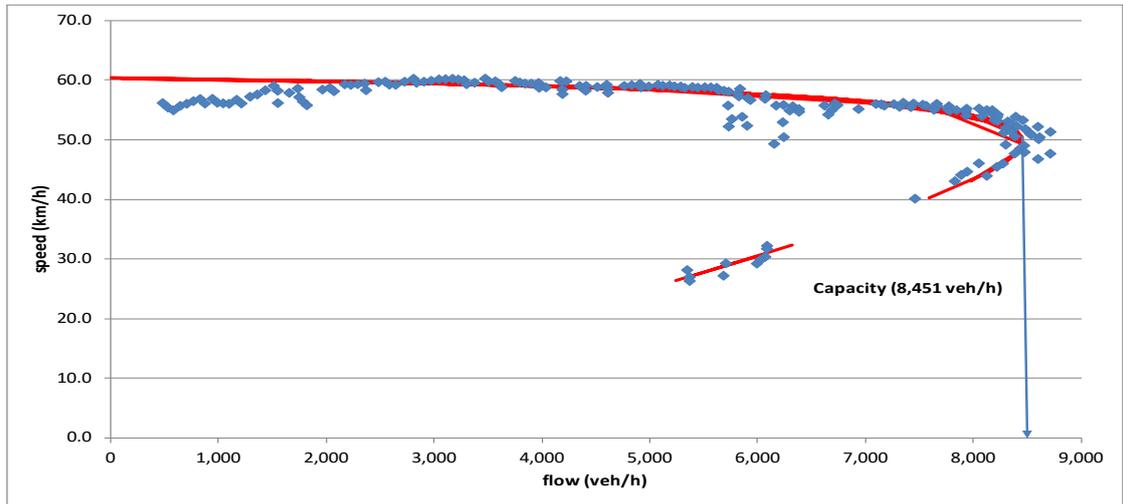
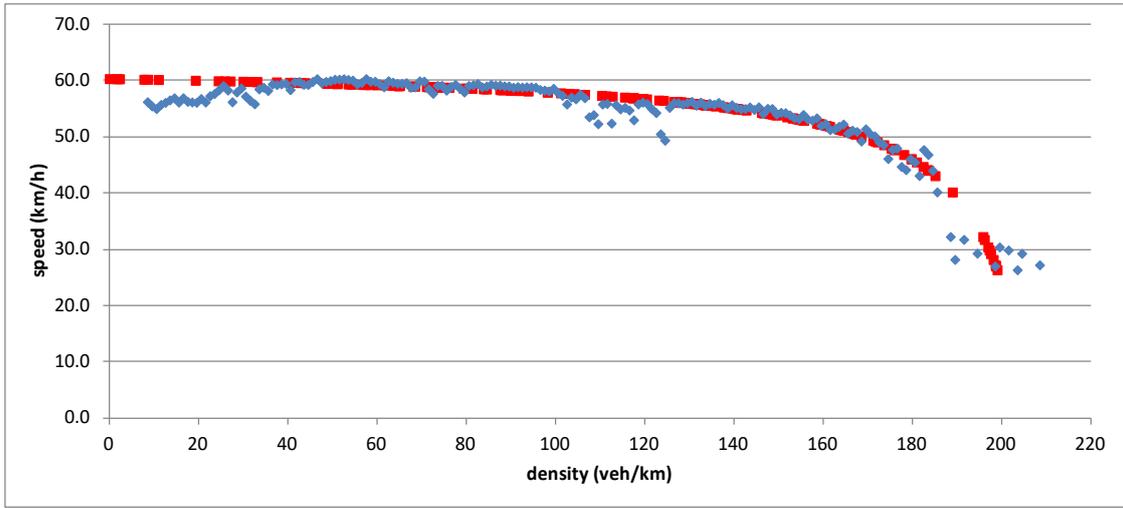
14779 IB Toorak to Yarra Blvd

'Series of bends (very tight radii around 350 – 600 metres) / geometrically tight; no shoulders and default 80 km/h speed limit' ( $s \leq 2.0\%$ )



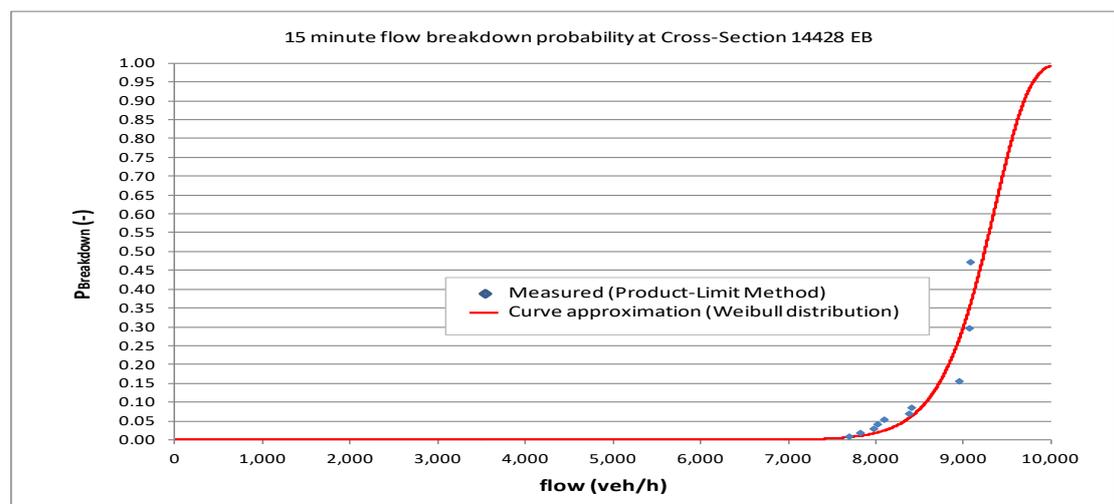
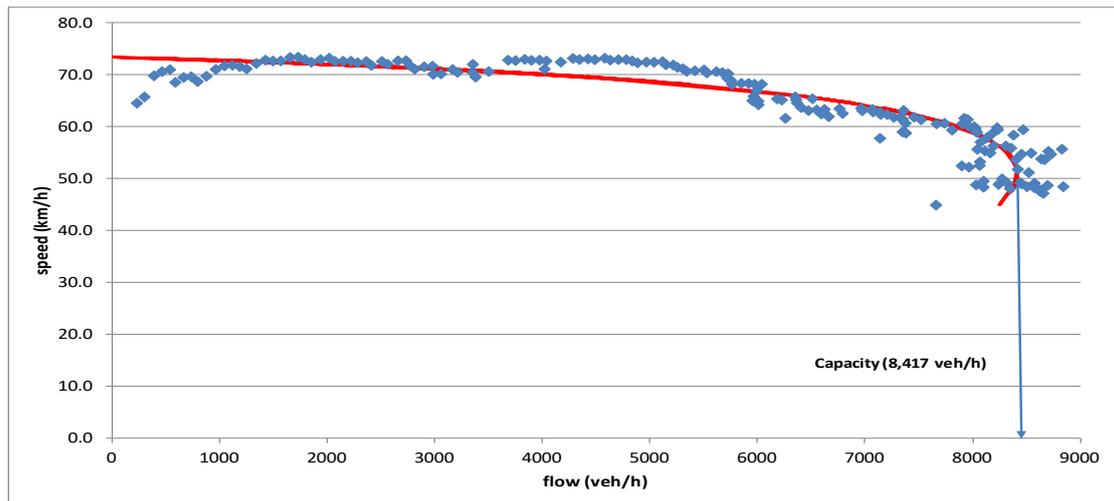
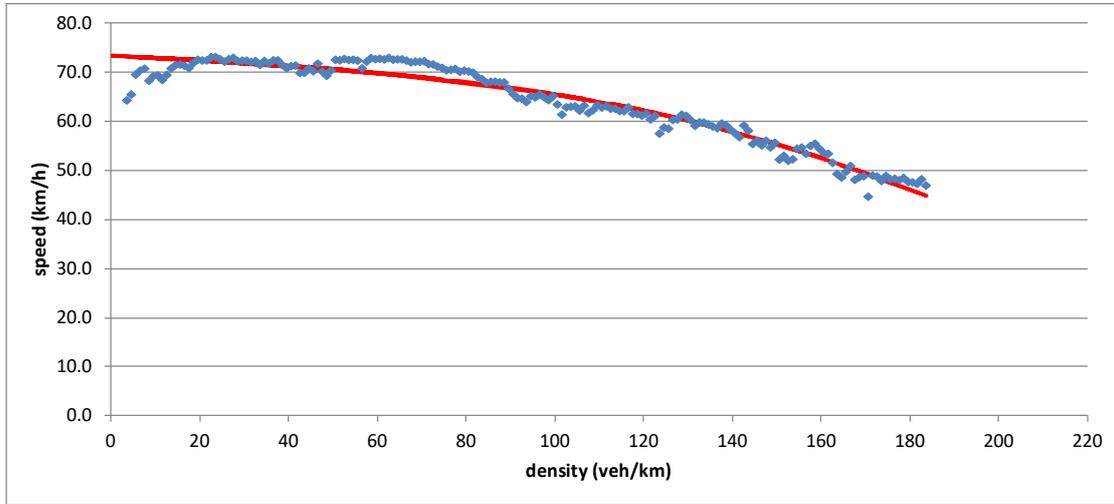
14427 WB Todd to Williamstown

'On the (eastern) approach to the West Gate Bridge, with a relatively long gradient' (s = 3.9% - on average over 1.3 km length; no shoulders and default 80 km/h speed limit)



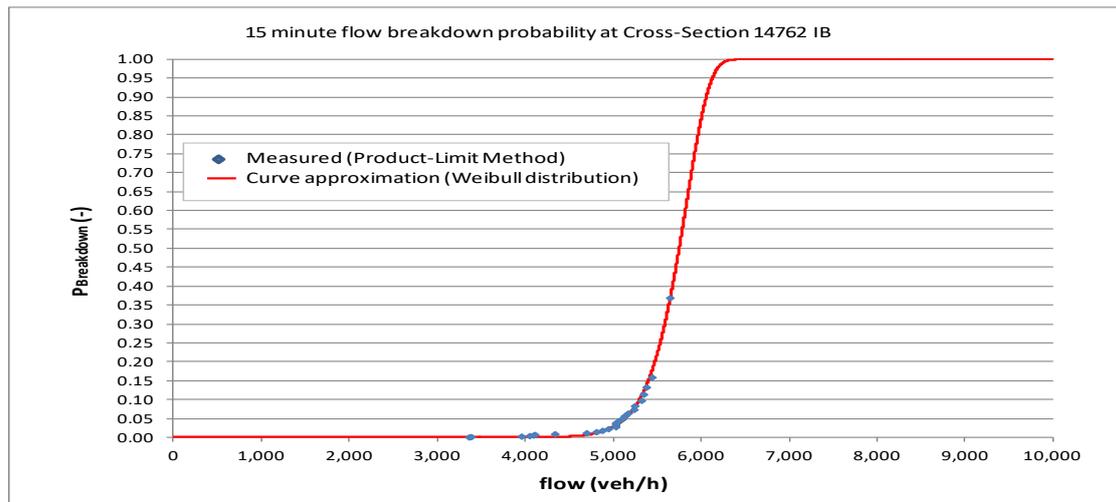
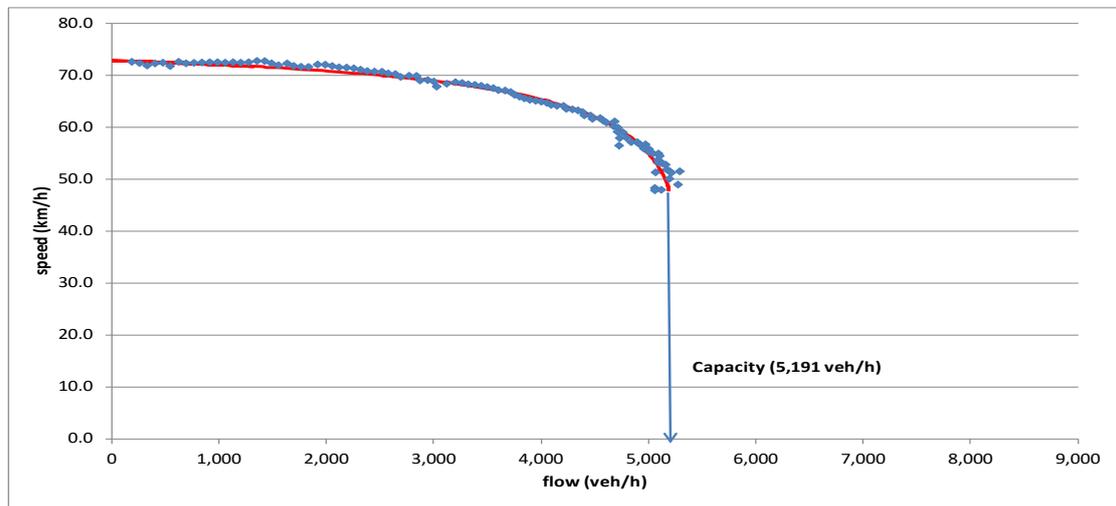
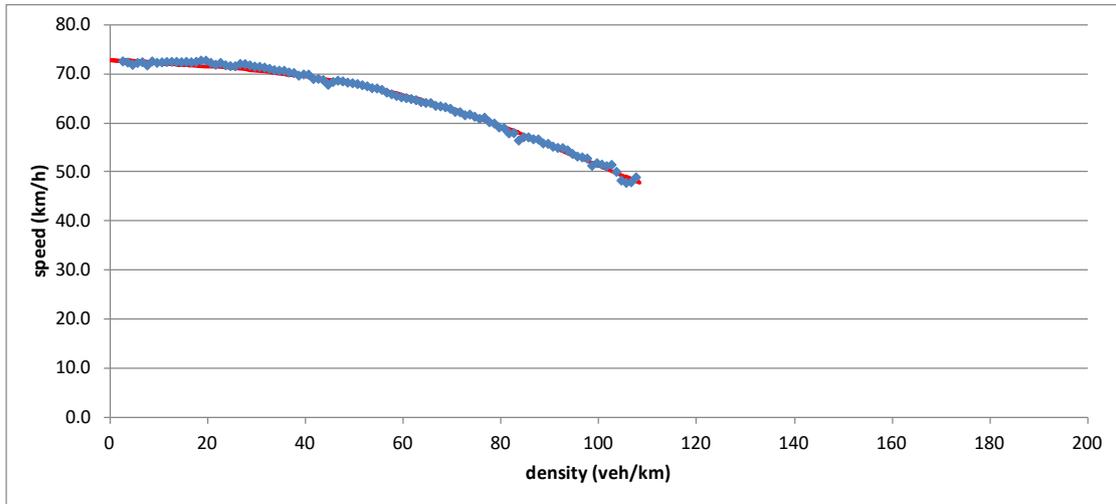
14428 EB Williamstown to Todd

'On the (western) approach to the West Gate Bridge, with a relatively long gradient' (s = 3.5% - on average over 1.2 km length; no shoulders and default 80 km/h speed limit')



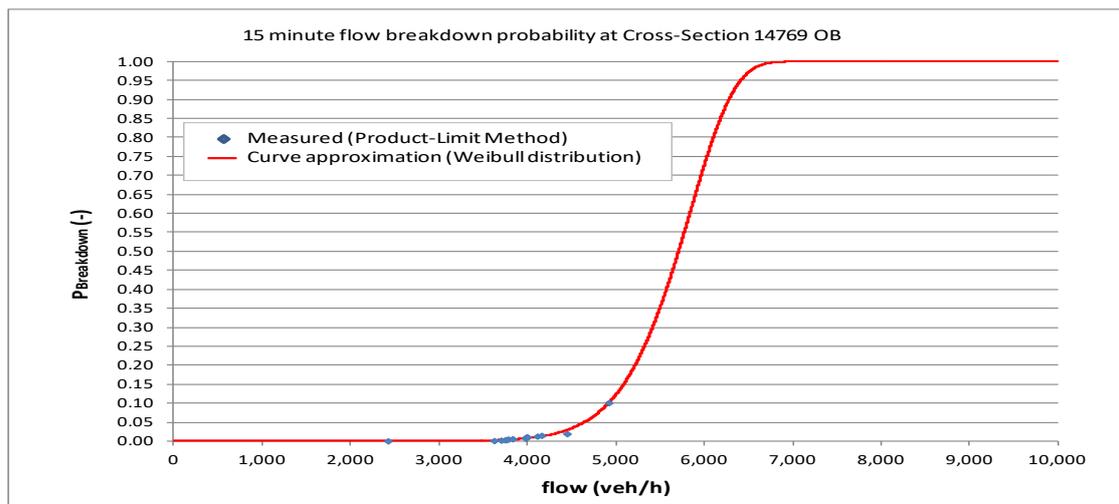
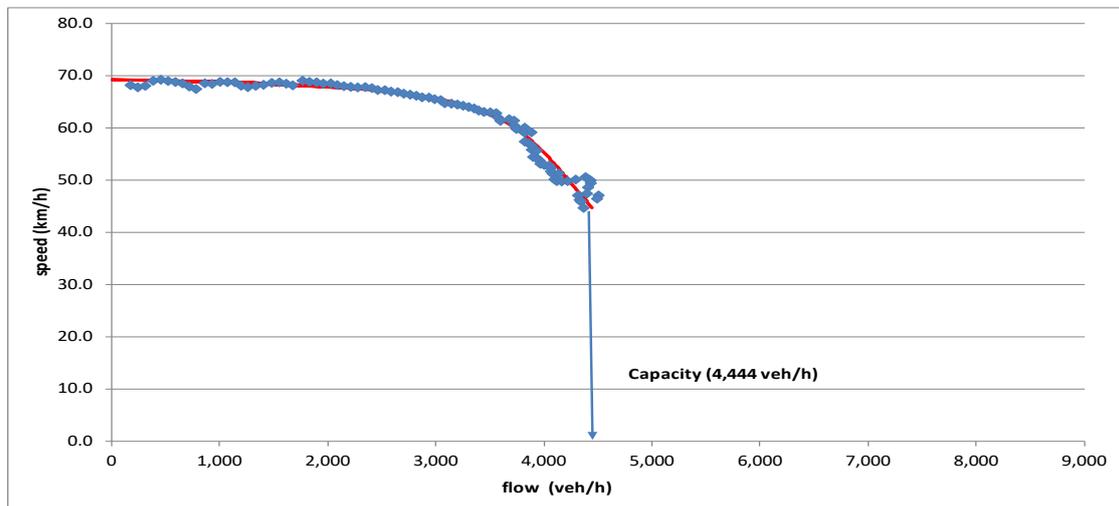
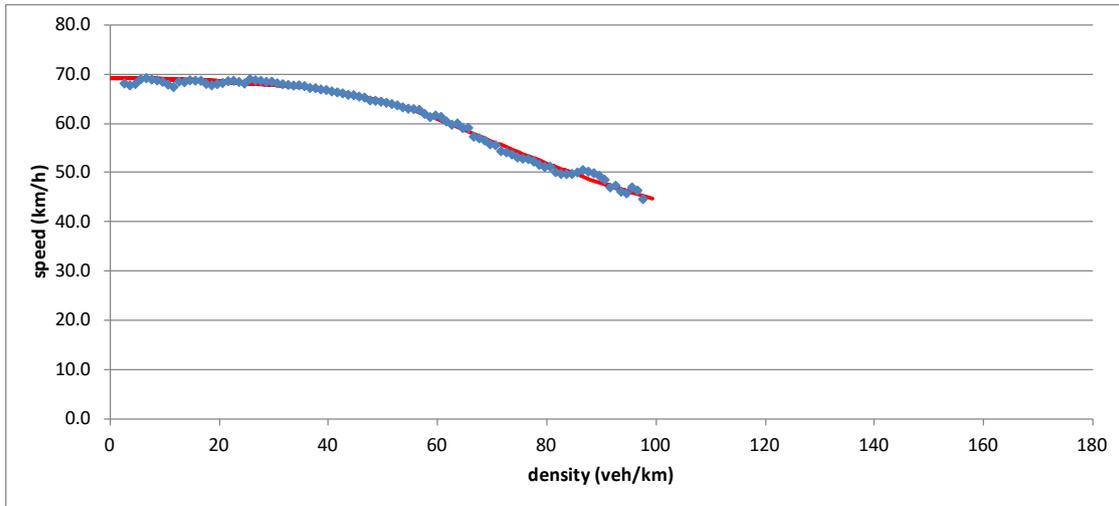
14762 IB Punt to Kings Way

Domain Tunnel



14769 OB Kings Way to Burnley

Burnley Tunnel



### 3.6 Comparison with International Design Values

The following tables show a comparison between Maximum Sustainable Flow Rates included in this Guide (compare to Table 2-1) with different international design values while using the corresponding highway capacity manuals (refer to (FGSV, 2015), (Henkens and Heikoop, 2015), (UK Highways Agency, 1999), and (Transportation Research Board, USA Highway Capacity Manual, 2016)).

The purpose of this comparison is limited to a plausibility check. The situation in other countries is not necessarily directly comparable to 'Motorways within large metropolitan cities' (refer to Section 3.2).

		Victoria	Germany	Netherlands	UK	US
A1	Deterministic pre- and post-breakdown (traffic flow model)	3,550 veh/h	3,800*0.9= 3,420 veh/h (Source: HBS 2015 - Table A3-2 for T100/T80/SBA (100/80km/h speed limit/'line control system'), inside urban areas)	NA	4,000 veh/h* (Source: Design Manual for Roads and Bridges (DMRB), TA 79/99 Amendment No 1 Traffic Capacity of Urban Roads – Table 2 - for 60mph speed limit, inside urban areas)	4,600 * 0.87 * 0.9 = 3,600 veh/h (Source: HCM 2016 - 'Capacity Under Base Conditions' for 60 miles/hr, adjusted based on Equation 12-10**)
A2	Stochastic pre-breakdown (censored and uncensored)	3,625 veh/h ***	NA	4,300*0.8=3,440 veh/h (Source: 'Dutch HCM' – Table 3.2 for 2 lanes)	NA	NA
<p>* Capacities may be used as 'starting points in the design and assessment'</p> <p>** Approximation: No direct calculation of the impact of XY% HGV on 'capacity' possible as Equation 12-4 actually used to increase demand</p> <p>*** Maximum Sustainable Flow Rate as included in Table 2-1:</p>						

**Table 3-4: Example 1 - 2 lane cross-section (inside urban areas)**

		Victoria	Germany	Netherlands	UK	US
A1	Deterministic pre- and post-breakdown (traffic flow model)	6,500 veh/h	7,550*0.9= 6,795 veh/h (Source: 'German HCM' - Table A3-2 for T100/T80/SBA (100/80km/h speed limit/'line control system'), inside urban areas)	NA	7,200* (Source: Design Manual for Roads and Bridges (DMRB), TA 79/99 Amendment No 1 Traffic Capacity of Urban Roads – Table 2 - for 60mph speed limit, inside urban areas)	9,200 * 0.87 * 0.9 = 7,200 veh/h (Source: HCM 2016 - 'Capacity Under Base Conditions' for 60 miles/hr, adjusted based on Equation 12-10**)
A2	Stochastic pre-breakdown (censored and uncensored)	6,775 veh/h ***	NA	8,200*0.8=6,560 veh/hr (Source: 'Dutch HCM' – Table 3.2 for 4 lanes)	NA	NA
<p>* Capacities may be used as 'starting points in the design and assessment'</p> <p>** Approximation: No direct calculation of the impact of XY% HGV on 'capacity' possible as Equation 12-4 actually used to increase demand</p> <p>*** Maximum Sustainable Flow Rate as included in Table 2-1:</p>						

**Table 3-5: Example 2 - 4 lane cross-section (inside urban areas)**

## Comments

- All values are for 15% HGV and  $s \leq 2\%$ .
- A1: Regarding the 'German HCM' values (HBS 2015 - Table A3-2), per lane capacities go up from 3 to 4 lane carriageways. The following should be noted: (1) The same 'capacity' determination methodology was used for this Guide and for the 'German HCM', (2) There are currently not many 4 lane carriageways in Germany, and (3) German conditions are different to those in Victoria in that interchange density is lower (with corresponding impacts on lane change intensity/friction which impacts on multiple lane carriageway 'capacity'), and the two inner lanes carry extremely high volumes since they are almost exclusively used by passenger cars.
- A1: Regarding the US HCM values, compare to VicRoads Managed Freeways – Freeway Ramp Signals Handbook, Section 2.4.1 (Figure 2.12).
- It should be noted that there are differences in the definition of 'capacity' (with corresponding implications on the potential application of a design factor 0.9):
  - German 'Handbuch fuer die Bemessung von Strassenverkehrsanlagen' ('German HCM'): 'Highest traffic flow that a traffic stream can reach at a cross section, under the prevailing road and traffic conditions.'
  - Dutch 'Capaciteitswaarden Infrastructuur Autosnelwegen' ('Dutch HCM'): 'Highest number of vehicles per time that can be expected to pass an average element of a lane or carriageway during a defined time period under the prevailing road, traffic, and operational conditions.'
  - UK DMRB: 'Maximum sustainable flow of traffic passing in 1 hour, under favourable road and traffic conditions.'

### 3.7 Adjustment factors used for non-standard conditions

The following adjustment factors from the Dutch HCM and the German HCM have been used to standardise measurement results and to calculate Maximum Sustainable Flow Rates for non-standard conditions.

#### HGV-Percentage

HGV Percentage	'capacity' multiplication factor <sup>1)</sup>
0%	1.15
5%	1.10
10%	1.05
15%	1.00
20%	0.96
25%	0.92
30%	0.88

1) Adopted from the 'Dutch HCM' (values are similar to the 'German HCM')

**Table 3-6: HGV-Percentage adjustment factors**

#### Gradient

Gradient	'capacity' multiplication factor <sup>2)</sup>
2%	1.00
3%	0.95
4%	0.90
5%	0.83

Adopted from the 'German HCM' (no values in the 'Dutch HCM')

- Relevant for length of gradient section  $\geq$  500 metres

**Table 3-7: Gradient adjustment factors**

### 3.8 Evaluation of Managed Motorway Technology

Besides use for planning, design and operation, the methodologies presented here should be a basis for the evaluation of investment in managed motorway technology which can be major (upgrade to managed motorway) or minor (re-tuning of the City Wide Coordinated Ramp Metering (CWCRM) or other algorithms and parameters).

Upgrades to managed motorways are often undertaken in the context of major civil works incorporating the addition of lanes. Hence, it is advisable to, along with assessment at carriageway level also do the pre- and post-project comparison on a by lane level, each of which should get done at bottlenecks as, for example, identified based on heat maps (compare to Figure 3-2).

Suitable approaches for the comparison of pre- and post-project performance are as follows:

- Probability of flow breakdown curves (compare to Figure 3-7 and Figure 3-8)
- Productivity frequency plot; i.e. histogram showing the frequency of low and high productivity intervals (refer to Figure 3-9 for an example)

A comparative 'capacity' analysis focusing on bottlenecks that were in similar locations in the before (unmanaged) and after M1 Upgrade (managed) situations was undertaken.

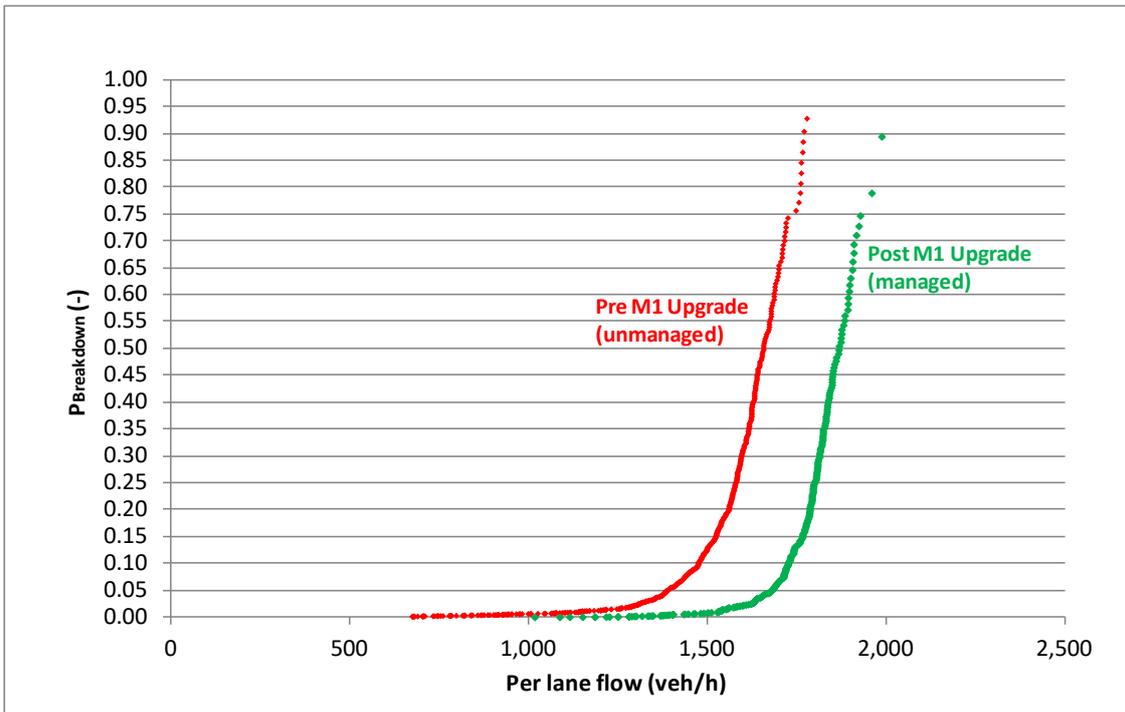
The 'capacity' values included in Table 3-8 were determined based on the same methodology as described in Section 3.3 and subsequently standardised to consider gradients  $\geq 2\%$  and HGV percentages other than 15%. Compared to the values included in Table 3-1, the measurement results presented here are equivalent to a 'capacity' reduction of around 15% (methodology corresponds to Sections 3.3.2 and 3.3.3, Approach 1).

Detector	Section from	Section to	Capacity	Number of lanes	Capacity per lane	Gradient	HGV percentage	Normalised Capacity per lane 1)
			veh/h		veh/h/lane	%	%	veh/h/lane
14304 IB	High	Burke	5,231	3	1,744	$\leq 2$	13	1,753
14312 IB	South Gippsland	Heatherston	5,172	3	1,724	$\leq 2$	11	1,658
7839 OB	High	Warrigal	4,407	3	1,469	$\leq 2$	13	1,440
7981 OB	Wellington	Jacksons	4,383	3	1,461	$\leq 2$	11	1,405
14316 OB	Belgrave-Hallam	Ernst Wanke/Tinks	3,230	2	1,615	$\leq 2$	11	1,553

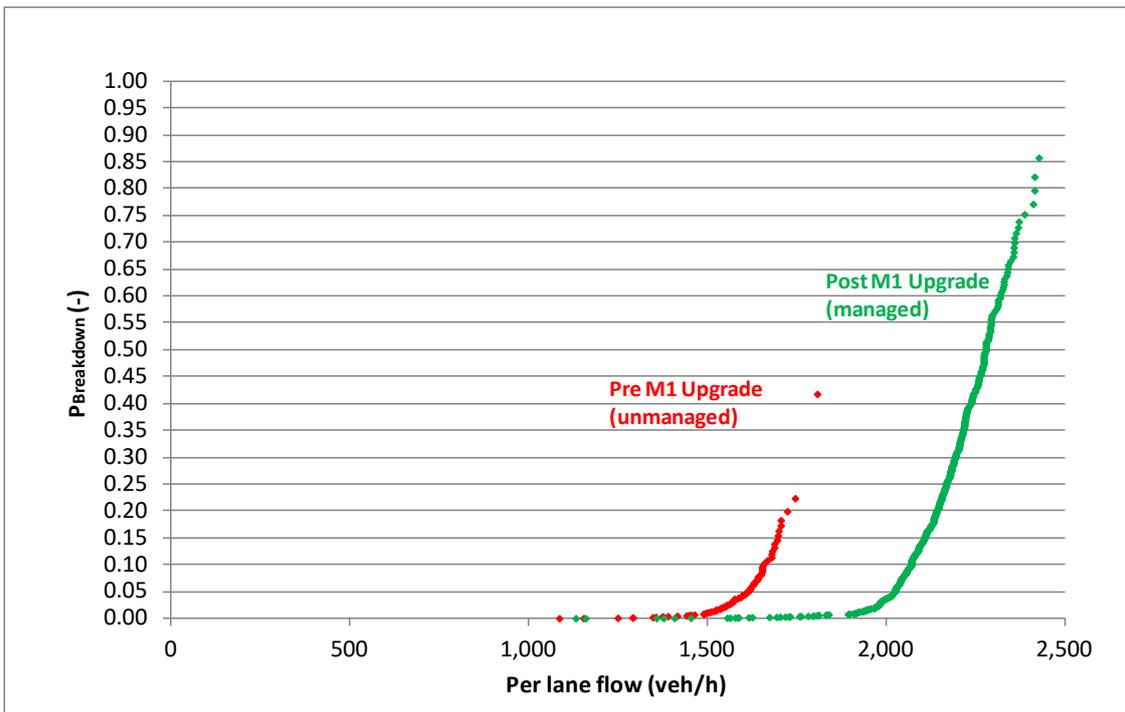
- 1) Considering gradient, HGV-percentage and lane changing conditions at measurement sites (normalised to  $s \leq 2\%$ , HGV = 15%; based on the factors shown in Section 3.7)

#### Table 3-8: Before M1 Upgrade capacities

Figure 3-7 and Figure 3-8 show a comparison of the probability of flow breakdown depending on the per lane traffic flow at the Warrigal Road and Hallam Bypass bottlenecks in the before and after M1 Upgrade situation. The methodology used corresponds to Section 3.3.2, Approach 2, Variant b). The graphs are based on data for the entire years 2007 and 2015. The curves illustrate that compared to the before situation a significant breakdown risk starts to occur at much higher flows in the after situation. This means that high traffic volumes can be managed much better with managed motorway technology. It should be noted that the avoidance of flow breakdowns (i.e. an abrupt decrease in average speeds) has a positive impact on safety as well as on 'capacity' (as per the above comparison), Maximum Sustainable Flow Rates and productivity.



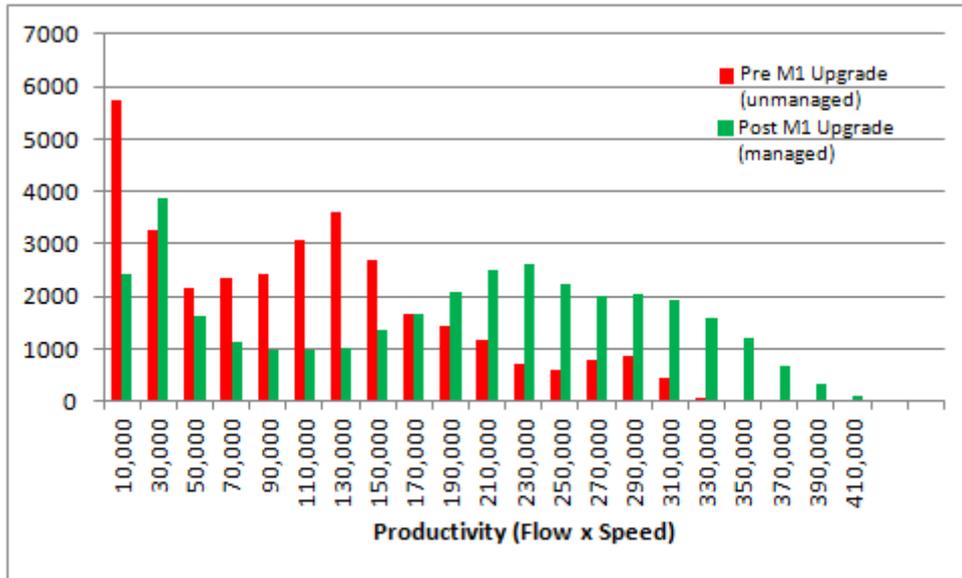
**Figure 3-7: Flow breakdown probability at Warrigal Road - before and after M1 Upgrade (7839 OB and 14547 OB, 15 min intervals)**



**Figure 3-8: Flow breakdown probability on Hallam Bypass – before and after M1 Upgrade (14316 OB and 14456 OB, 15 min intervals)**

The comprehensive data analysis undertaken also revealed that the so called ‘capacity drop’ (i.e. the decrease in speed and throughput due to a flow breakdown) is lower in the after M1 Upgrade situation.

The differences in the flow breakdown probability distributions and ‘capacity’ drops combine to have the overall difference of the hourly ‘capacity’ values of around 15%.



**Figure 3-9: Example for a Productivity Frequency Plot (14316 OB and 14456 OB, 15 min intervals)**

As a plausibility check, the above result was verified based on a comparison of per lane throughput in the before and after the M1 Upgrade situations which were in the order of 16-19% (compare to Managed Motorways Framework, Appendix – Table A1).

### 3.9 Influence of tight curves

Numerous studies show a link between tight horizontal curves and increasing crash rates (e.g. (Khan et al., 2012)). However, the impact of tight curves on ‘capacity’ is largely unexplored and difficult to measure. This is because there are opposing influences on road user behaviour which may even neutralise each other in some situations:

Based on observations across the motorway network, one can reasonably postulate that:

- Tight curves reduce discretionary lane changing. The nature of a tight curve is such that even if vehicles are travelling at the same speed in each lane, the progression is further on the inside of the curve, resulting in differential lane progression, thereby reducing the ability to lane change (the more lanes, the greater the effect);
- reduced lane changing means reduced operational efficiency (since they are needed to load and unload the motorway); and
- reduced lane changing also means reduced ‘friction’ – with corresponding impacts on ‘capacity’.

The measurement results summarised in Section 3.5 do not show an influence of the curve located between Toorak Road and Yarra Boulevard on ‘capacity’ (Centre line radius: 1,170 feet = 356.62 metres – Source: (Melbourne and Metropolitan Board of Works, 1968)). Extended analysis of the ‘tight curve problem’ also looked at the Eastern Freeway section between Bulleen/Thompsons Road and Doncaster Road: The corresponding curve with a radius of down to 450 metres (Source: VicRoads Spatial Database) in a 100 km/hr operating environment does not significantly reduce the ‘capacity’ of this section. Mainly based on the analysis of ‘Heat plots’, the analysis concluded that the Bulleen/Thompsons Road On-Ramp (rather than the 450 m Radius curve) is usually the bottleneck here.

The situation in this location can be summarised as follows:

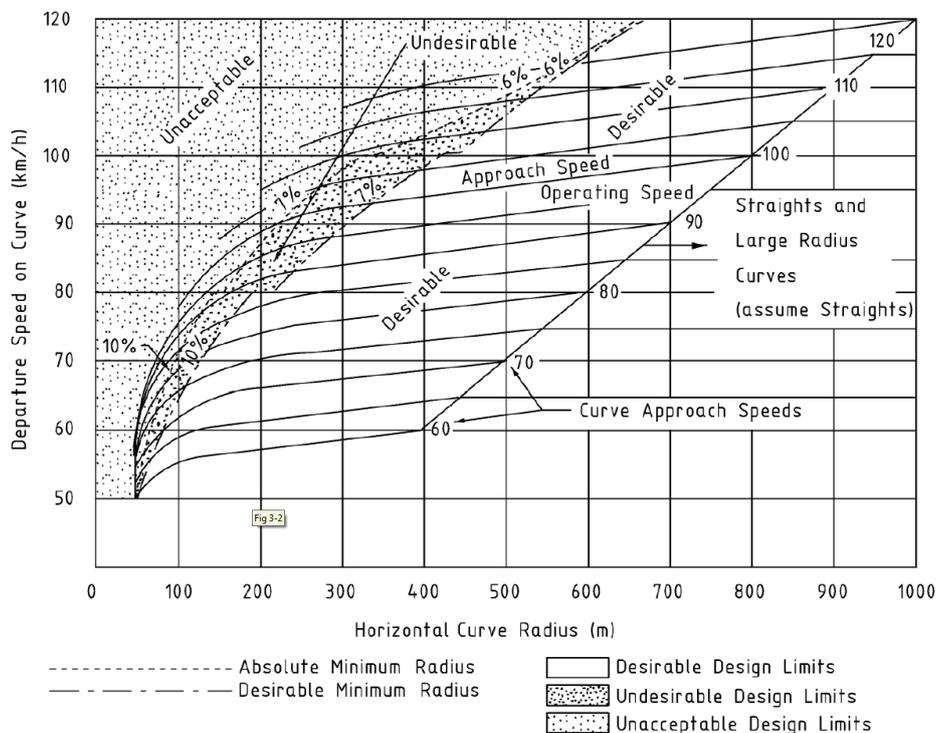
- The Bulleen Road on-ramp merge creates a bottleneck in advance of the curve;
- The flows released from this bottleneck may still be reasonably high given the differential flows across lanes;
- As traffic accelerates away from the merge bottleneck, headways open up, allowing the necessary lane changing to occur for the downstream exit; and
- The tight curve (combined with the differential lane progression) then discourages excessive discretionary lane changing.

From a **productivity** perspective, there is clear evidence that tight curves have a negative impact because of their speed reducing effect. For example, the Austroads 'Guide to Road Design Part 3: Geometric Design' includes a 'Car deceleration on curves' graph which allows the designer to estimate the speed to which a vehicle may decelerate to or maintain when entering a curve of a given radius (refer to Figure 3-10). For an approach speed of 100 km/h, curve departure speeds start to slowly deteriorate from a curve radius of 800 metres downwards, with corresponding impacts on productivity. Although for a curve radius of for example 600 or 550 metres the reduction in speed is relatively minor (3 or 4 km/h respectively), given the high vehicle numbers, the overall impact on productivity is already very high.

The NCHRP Report 783 'Evaluation of the 13 Controlling Criteria for Geometric Design' (Harwood et al., 2014) in Section 4.4 (Figure 8) also includes a graph showing the speed deterioration caused by curves (refer to Figure 3-11).

VicRoads measurements on the Eastern Freeway between Bulleen/Thompsons Road and Doncaster Road confirmed the impact of tight curve radii on speed and therefore productivity. From a traffic efficiency perspective, it is therefore not recommended to adopt curve radii less than a 750 metre threshold, irrespective of any potential negative impacts of tight curves on 'capacity'.

It is intended to do more practical analysis work on the 'tight curve problem' in relation to impacts on 'capacity' in the future.



**Figure 3-10: Car deceleration on curves (Austroads, 2016)**

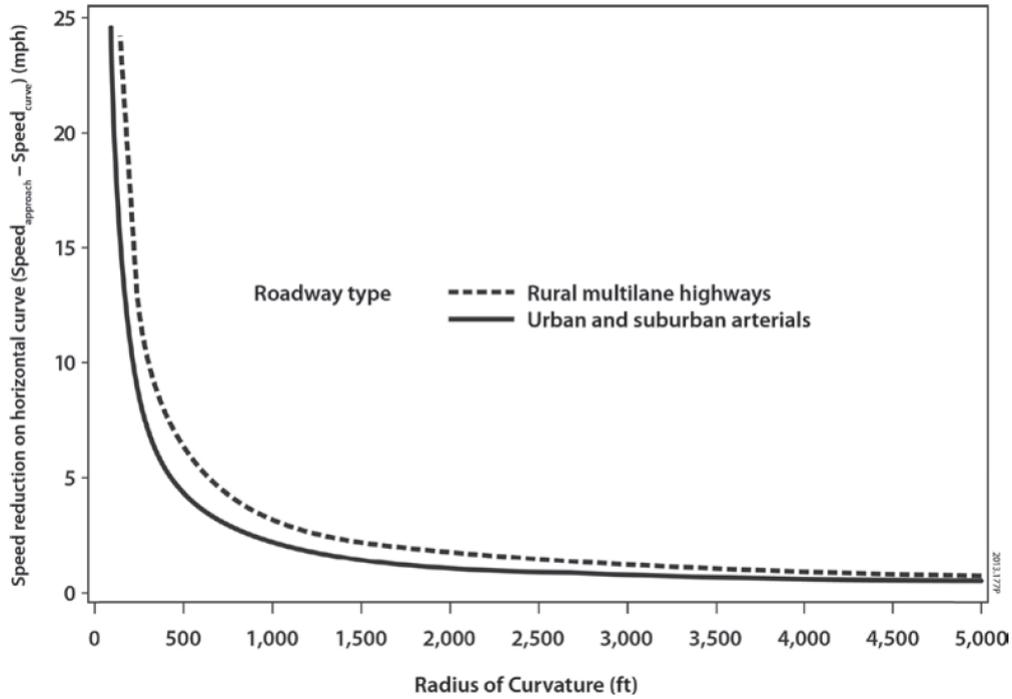


Figure 3-11: Speed reduction on curves (Harwood et al., 2014)

### 3.10 Future Investigation Work

Areas that are either under investigation at present or may be explored further in the future include:

- ‘Capacity’ values for merging, diverging and weaving including at complex auxiliary lanes;
- More detailed analysis of the differences between unmanaged and managed motorways; this comprises ‘capacity’, productivity, probability of flow breakdown and the different traffic flows after a flow breakdown (‘capacity’ loss and recovery path);
- Link between congestion and crash rate (continuation of research based on VicRoads data); and
- The impact of tight curve radii on ‘capacity’ and productivity.

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