Highspeed Railway Assessment - Rail Specific Planning and Development Analysis

APPENDIX 2

REPORT

Design of Crossing loops

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Abstract

Crossings are important events on single-track railway lines. The crossings affect the travel times, the risk of delays and the capacity. This report have two main objectives: to analyze capacity for mixed traffic on single-track lines and to make a deeper study of single-track lines designed for homogeneous high-speed traffic.

Capacity for mixed traffic

Mixed traffic is a challenge already on double-track lines where faster trains catches up with slower ones. For single-track lines this phenomenon is combined with crossings which makes it even more difficult to find feasible timetable solutions. Together these operational characteristics limit and restrict capacity and timetable flexibility on single-track lines operated with mixed traffic. Generally the frequency of service turned out to be the most important and constraining factor. Based on the performed analyses we want to make the following comments:

For high-speed traffic mixed with regional traffic:

- Timetable flexibility is generally very low for this kind of traffic. It is difficult to change or develop the traffic
- A frequency of 0.5 high-speed trains/h/direction and 0.5 regional trains/h/direction will work on a normal[1] single-track line.
- A combination of 1 train/h/direction for one of the train types and 0.5 trains/h/direction for the other will probably work on an adjusted[2] single-track line. We did not manage to verify this since more work has to be put on how to adjust the infrastructure.
- The inter-loop distances should be chosen with intended speed ratios and frequencies of service in mind.

For high-speed traffic mixed with freight traffic:

- Timetable flexibility is low also for this traffic mix.
- A frequency of 0.5 high-speed trains/h/direction and 1 freight train/h/direction will work on a normal[1] single-track line.
- A frequency of 1 high-speed train/h/direction and 0.5 freight trains/h/direction will also work on a normal[1] single-track line with an average inter-loop distance of 10 km.

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[1] A normal line is not constructed with a special timetable in mind. It is rather adjusted to manage timetable changes and development.

[2] An adjusted line is a line where certain parts are constructed for time efficient crossings and overtakings, e.g. partial double-tracks, loops with four tracks for combined crossings and overtakings etc.

Capacity for homogeneous high-speed traffic

- The homogeneity increases the capacity since no catch-up effects occur. It is also easier to adjust the infrastructure for time efficient crossings since the number of crossing areas is lower.
- A frequency of 1 high-speed train/h/direction will work on a normal single-track line.
- A frequency of 2 high-speed trains/h/direction will probably work on an adjusted single-track line. We did not manage to verify this since more work has to be put on how to adjust the infrastructure, but the special analysis of homogeneous high-speed traffic supports this supposition.

Time supplements and average speeds for homogeneous high-speed traffic

The second part of this report deals with homogeneous high-speed traffic and the amount of time supplement needed in the timetable for the crossings. Calculation of these supplements is an essential part of the estimation of achievable average speeds and travel times.

Only cases with high punctuality, homogeneous traffic, high standard infrastructure and limited frequency of service are evaluated. The results show that a single-track high-speed line is a realistic alternative, provided that these requirements are met.

Three kinds of crossing loop are to be considered in further investigations:

- **Scheduled crossing loops with regular passenger stop**. These loops is most preferable designed with turnouts for 160 km/h and extended loop tracks of 1 500 m. The passenger stop is then located at braking distance from either loop end, i.e. close to the other end of the loop. This design minimizes the amount of time supplement needed for the crossing. In practice, turnouts located close to the stop position do not necessarily need to be given a higher standard than 100 km/h since this speed restriction does not affect trains accelerating from a stand still.
- **Scheduled crossing loops without regular stop.** These loops need to be extended into partial double-tracks. A recommended length is 20 000 m. Turnout standard should be 160 km/h. A trapezoid design is preferable or a rhomboid.
- **Secondary crossing loops.** These loops are not planned to be used as scheduled crossing loops. They are rather reserve loops to be used in great delayed situations and for extra trains operated in rush-hour traffic. It is however important that also these loops have a high standard, 160 km/h turnouts and 1 500 track length, since the uncertainty of future operation and timetable calls for timetable flexibility, i.e. possibility to find alternative feasible timetable solutions than the one used to for the infrastructure design.

The results are a good base for further evaluation with studies of real alignment, loop locations etc.

Introduction and background

This report deals with infrastructure design of single-track sections of a future Norwegian high-speed railway network. Several different design variants are tested for several operation strategies. The evaluation is divided into two independent parts:

- Capacity analysis.
- Delay analysis and evaluation of required time supplements for crossings.

The capacity analysis is a general timetable analysis where different infrastructure designs are tested and evaluated for different traffic conditions. A combinatorial and generic algorithm (TVEMS) is applied to determine the capacity for different traffic mixes on different infrastructure designs. The section clearly shows the capacity limitations for a single-track line operated with a mix of high-speed trains and slower trains. It is shown that the mix of different speeds limits the capacity severely.

The second section concentrates on homogeneous high-speed traffic, or low frequent mixes of high-speed and slower traffic. The aim of this section is to find requirements to achieve short and reliable travel times for this kind of operation. Generally, these requirements are difficult to meet on single-track lines due to time consuming crossings of opposing trains. Each crossing is also connected to a risk of delay propagation.

It is, however, possible to operate high-speed traffic on single-tracks under special conditions. Some of these conditions are:

- **High punctuality**. Great delays must not be very frequent.
- **Homogeneous traffic**, i.e. no or low speed differences between trains.
- **Stable timetable**. Single-track sections imply restrictions on the timetable which limits the number of possible timetable variants that meet travel time requirements etc.
- **High standard crossing loops**. The crossing loops must be reliable, have a high availability and be designed for time efficient crossings.
- **Limited capacity utilisation**. The number of scheduled trains on each single-track section must not be too high.

The second section addresses operational situations where these conditions are met. This means that the line sections analyzed are operated with completely homogeneous traffic, i.e. regular timetable on a dedicated line sections. The frequency is assumed to be 1-2 trains/h and direction.

Only high-punctuality operation is considered. This is a reasonable assumption for modern, dedicated lines with homogeneous traffic. The evaluated infrastructure designs are chosen to have a high standard to give low time consumption in crossing situations and high timetable flexibility. This is meant to limit the restrictions and drawbacks implied by the single-track sections as much as possible.

Capacity on single-track railway lines with mixed traffic

Effects of speed differentiation on single-tracks

The two-way traffic that usually occurs on single-track lines limits and restricts capacity. The constraints increase when the traffic is heterogeneous. This is clearly illustrated by the timetable example in figure 1. The figure shows low frequent high-speed traffic at 300 km/h mixed with equally low-frequent regional traffic at 200 km/h.

The speed differences create wedges that make it difficult to find feasible, not to time-consuming crossing patterns in the timetable. The wedges are extra constraining when they are repeated like this in a periodic timetable.

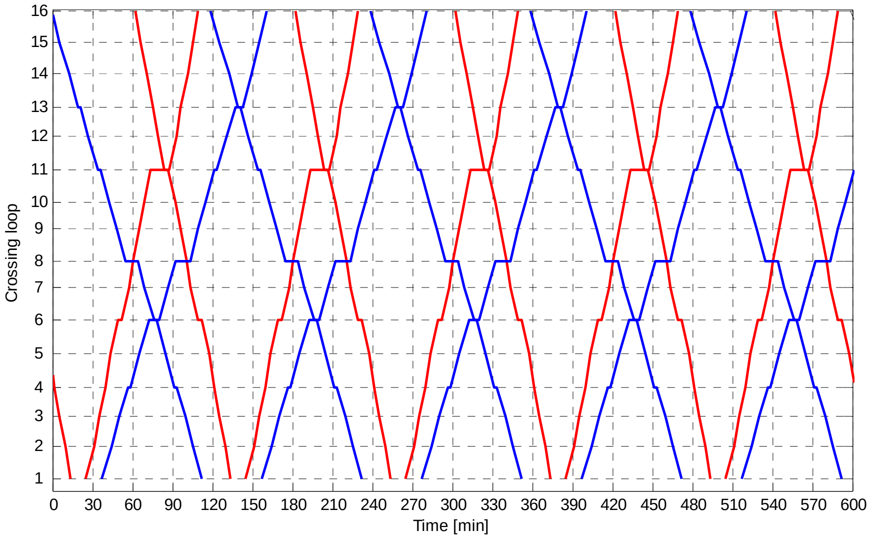

Figure 1 Timetable example for mixed traffic on a single-track line. High-speed trains red and regional trains blue. The shown line is about 300 km long and with an average inter-loop distance of 20 km.

The speed differences also induce overtakings where the slower trains are passed by the faster ones. Both crossings and overtakings give scheduled delays to the trains and risk of delay propagation. This means that the number of crossings and overtakings have to be limited in order to reach a required average speed[3].

The number of needed crossings and overtakings depends on the line length, the speed levels and the frequencies of service. This means that the frequency of service cannot be too high for a given line design, speed levels and requirements on the average speed. This is also the major result presented later and the reason why passenger traffic that is so dense that overtakings occur frequently is not of commercial interest.

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[3] It is possible to adjust the infrastructure for time efficient crossings. This is presented in the second section of this report.

TVEMS – a short model description

TVEM, Timetable Variant Evaluation Model for Single-tracks, was constructed to evaluate the impact of infrastructure and timetable factors on Swedish freight lines. The model is deterministic and generic. The fact that (most) passenger traffic is operated with periodic timetables is used to generate a large number of timetable variants. The spare capacity between the periodically operated traffic is then evaluated. Other features that might be evaluated are scheduled delays and utilisation of different passing loops.

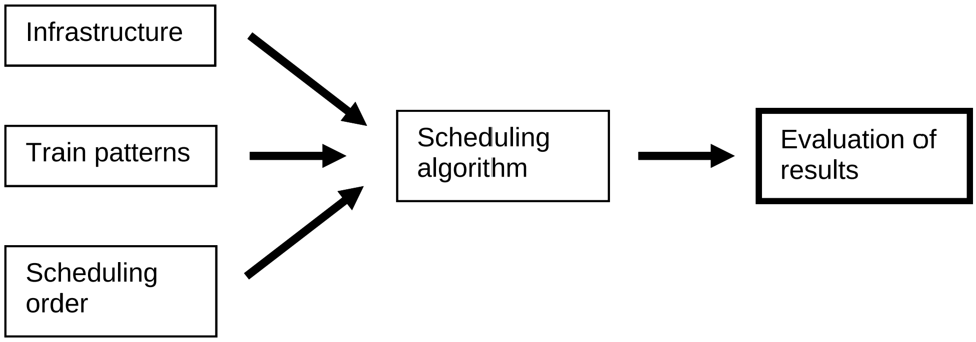


Figure 2 Structure of TVEMS.

The scheduling algorithm is the heart of the model. Here, the train patterns are systematically scheduled according to their pre-defined (priority) scheduling order. Crossings and overtakings are introduced to resolve conflicts. Both block section between crossing loops and loop tracks within the loops are modelled as capacity resources allocated by the trains throughout the scheduling procedure.

All results are presented as distributions since different timetable variants result in different capacity, scheduled delays etc. The impact of the timetable design is thus displayed directly. It is also possible to vary the infrastructure design automatically in TVEMS, and the impact of distances between passing loops can therefore also be evaluated.

TVEMS was developed for analysis of ordinary single-tracks, i.e. without partial double-tracks and other measures that aim at decreasing time loss in crossing situations to reach a higher average speed. The model his therefore just used for capacity analysis within this assignment. These analyses are complemented by a detailed analysis of crossing courses and delay propagation in the second section.

Methods like TVEMS are especially useful in planning stages where the timetable, i.e. frequency of operation, vehicle characteristics, stopping patterns etc, is unknown and one or several alternative infrastructure designs are to be evaluated.

In this project TVEMS is applied to evaluate the effect of rather complex four-pattern cases, i.e. operation of two service type per direction. The high-speed trains are given priority over the secondary trains (regional or freight trains), which means that all high-speed trains in one direction are scheduled first without any scheduled delays. The timetable is then completed with high-speed trains in the opposing direction followed by the secondary trains in the first direction and then last the secondary trains in the second direction. Capacity conflicts are resolved by introduction of crossings and overtakings on crossing loops. TVEMS is further described in Lindfeldt (2011).

The analysed factors, i.e. inter-loop distance, speed level, frequency of service and traffic mix, all affect the crossing and overtaking possibilities which in turn affects the measure of performance such as capacity etc.

Measures of performance

There is no evident way to measure capacity. The situation is even more complicated since the two traffic mixes analyzed within this assignment call for different measures of performance. This is because regional trains are operated according to a periodic timetable (whose frequency is strongly correlated to the frequency of the high-speed trains), whereas freight trains are operated one-by-one in non-periodic patterns.

Regional traffic

One measure of performance was chosen for the regional traffic:

- Ability to find a feasible timetable.

This is actually a Boolean measure, since it is either 0 or 1. The reason for this simple measure is that a single-track line operated with mixed traffic turned out to be very sensitive to the frequency of service at which the trains are to operate. This results in a very clear difference between a total traffic volume of one and two trains/h and direction. A traffic limited to one train/h and direction (every second is a high-speed train) gives numerous of feasible timetable solutions, whereas two trains/h and direction cannot be scheduled unless the infrastructure is adjusted for more time efficient crossings (double-track sections).

Freight traffic

The freight traffic is different, since it is not operated according to a periodic timetable. The requirements for high average speed are also somewhat lower for freight trains than for regional trains. It is therefore easier to find feasible timetable solutions for this mix and it is natural to use a common capacity measure for the mix of high-speed and freight trains:

- Number of (possible) freight trains/h and direction.

The number of freight trains/h shows the spare capacity available in between the high-speed trains. Even if the line is not planned for this kind of dense freight traffic, this measure is a good indication of the capacity that can be used for other purposes than high-speed services. The standard deviation of number of freight trains is complementary and gives an idea about how sensitive the calculated capacity is to the exact inter-loop distances.

Independent factors

Four independent factors are studied within this assignment:

- Traffic mix.
- Average inter-loop distance.
- Maximum speed for high-speed trains.
- Frequency of service for passenger trains.

All factors are modelled on two levels. This means that 16 unique variants are tested.

Two different **traffic mixes** are evaluated:

- High-speed and regional trains.
- High-speed and freight trains.

The **average inter-loop distance** is an important factor. Two levels are tested: 10 and 20 km distance. The distance is here defined as the length of the double-track section between two adjacent passing loops. Real railway lines do not show a constant inter-loop distance. In order to simulate these fluctuations, each distance was randomized within an interval of the specified inter-loop distance ± 3 km.

This procedure gives a somewhat randomized design, which in turn calls for replicates, i.e. test of several line designs. For this reason 100 line designs were sampled for each of the two tested inter-loop distances.

The **maximum speed** of the high-speed trains is another important factor. The higher this speed is, the more often do crossings and overtakings between trains occur. Two levels of maximum speeds are evaluated: 250 and 300 km/h. The vehicle characteristics for these trains are shown in appendix 1.

The **frequency of service** is another factor that directly influences the crossing and overtaking pattern and the use of and need for crossing loops. We assume and evaluate two levels: one high-speed train every second hour and one high-speed train per hour. Periodic operation (regular timetable) is presumed in both cases. For simplicity the regional trains are assumed to be operated at the same frequency as the high-speed trains.

In the case where the traffic mix consists of high-speed and freight trains, no special frequency is assumed for the freight trains. Instead, these trains are used to “fill up” capacity in between the high-speed trains to get a measure of the spare capacity.

Constants

Several factors have been modelled as constants. The most important are:

Line length. The total line length was set to ca 300 km. A tolerance of ± 15 km was applied to allow the randomization of inter-loop distances to work properly.

Loop design. The loop design was chosen i accordance to the passing loops analysed in another part of the assignment, i.e. with entrance turnouts allowing 130 km/h in the diverging track and a total loop length of 1 060 m. Each loop was given two loop tracks to allow combined crossing and overtaking. This standard is probably not needed for every loop, but at this investigation stage it is important not to exclude timetable variants due to lack of loop tracks. Our experience says that a third track is very useful at some loops, but not necessary everywhere. A deeper analysis has to be performed to decide where to construct extra loop tracks.

The minimum **block length** was chosen to 5 000 m. This means that each inter-loop distance was divided into block sections of 5-10 km length each. The signalling system is assumed to be of ERTMS level 2 type.

Headway. The minimum headway, used for scheduling, was set to 240 seconds.

Accepted scheduled delay. The maximal accepted scheduled delay is an important constant. It is used to judge whether a train path is to be accepted or rejected. Too much scheduled delay means that the average speed of the service drops below the limit for market attractiveness. This is especially important for passenger services, but also freight services must not get too low average speeds. In this assignment we used an upper limit of 15% for the passenger trains and 40% for the freight trains. These values are set to rather high values due to the expected numbers of crossings and overtakings to be scheduled. Special measures, such as partial double-tracks, may be taken to decrease the realised scheduled delay.

Dwell times. The scheduled dwell times for passenger stop were assumed to be 120 seconds for high-speed services and 60 seconds for regional services.

Vehicle characteristics for regional and freight trains. The regional trains were operated with a maximum speed of 200 km/h. More detailed vehicle characteristics are shown in appendix 1. The freight train was a light freight train of 560 tons and maximum speed of 140 km/h. All vehicles have a deceleration of 0.8 m/s^2 which helps to make the overtaking courses time efficient.

Step length in scheduling. The step length in the scheduling procedure was one minute. This means that one hour has 60 theoretical time shifts, i.e. time locations for a train path.

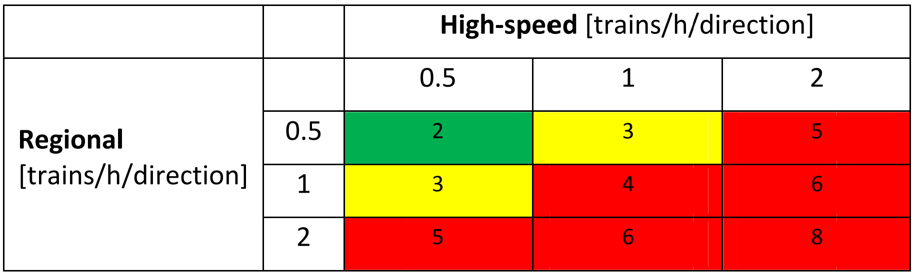
Results

High-speed traffic mixed with regional traffic

The study of mixed high-speed and regional traffic focuses on the effect of inter-loop distance, maximum speed of the high-speed trains and frequency of service. A simple pre-study with TVEMS showed that the frequency factor is so important that it needs to be addressed in special order. Table 1 shows this.

The results from the initial evaluations showed that timetables for mixed traffic on single-track line are not easily found. A mix consisting of one high-speed and one regional train every second hour and direction is possible (green cell in table 1). However, a deeper analysis of this frequency combination shows that timetable flexibility is limited, see below.

The capacity is insufficient if either the high-speed or the regional traffic is to be operated at a frequency of one train/h/direction, see yellow cells in table 1. It is however likely that some adjustments, i.e. partial double tracks etc, in the infrastructure would increase capacity enough to make these frequency combinations possible.



		High-speed [trains/h/direction]		
Regional [trains/h/direction]		0.5	1	2
	0.5	2	3	5
	1	3	4	6
	2	5	6	8

Table 1 Feasible frequency combinations. Red: no feasible timetables found. Yellow: no feasible timetable found, but feasible timetables are likely if the infrastructure is adjusted to a specific timetable design. Green: a small number of feasible timetables found. Numbers within each cell shows total traffic load, i.e. number of operated trains/h on each line section.

Figure 3 illustrates the capacity problems that occur when a line shall be operated by one high-speed train every second hour and one regional train every hour and direction (one of the yellow cells in table 1). TVEMS could not find feasible train paths for the up-bound regional trains (black arrows in figure 3). The reasons for this are obvious when the number of additional crossings, and probably also overtakings, is counted.

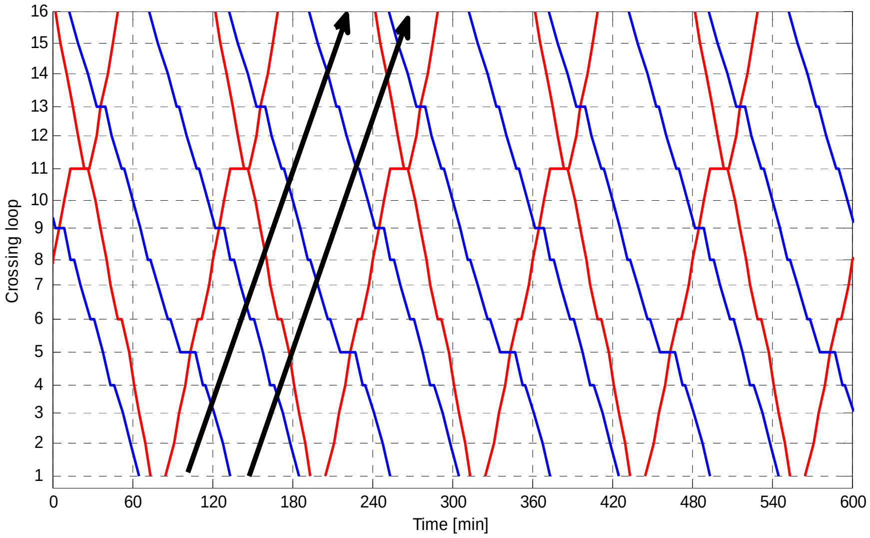


Figure 3 Timetable example with high-speed trains every second hour (complete timetable) and regional trains once an hour in one direction (incomplete timetable). The difficulties to fit in the missing, up-bound regional trains is obvious (black arrows).

This pre-study can be summarized as follows:

- Mixed traffic is very capacity consuming. The frequency of service has a major impact. The difference in needed capacity for 0.5 and 1 train/h/direction is great. An intermediate frequency, say 0.7 trains/h/direction, would be more suitable from a technical point of view. This is however not commercial and hence not to be regarded.

- A frequency of 0.5 trains/h/direction for high-speed and regional traffic is a feasible frequency combination for an ordinary single-track line.
- The frequency for either high-speed or regional traffic might be increased (yellow cells in table 1). But this requires the infrastructure to be designed for a predefined timetable.

Despite the major impact of the frequencies of service, it is of interest to analyze how the other two factors, inter-loop distance and maximum speed for high-speed trains, affect the ability to find feasible timetables.

One hundred infrastructure designs were sampled, according to the constraints described above, with 10 km and 20 km inter-loop distance respectively and TVEMS was applied to find feasible timetables on these. The frequency of service was set to 0.5 trains/h/direction for each train type and the scheduling was performed both for maximum speed for high-speed trains of 250 km/h and 300 km/h respectively.

Each feasible timetable contains four train patterns: high-speed up/down-bound and regional up/down-bound. This means that the theoretical number of timetables is $1*120^3$ = 1.73 million. All these timetables would be feasible if the line was quadruple track, i.e. if the four train pattern could be scheduled totally independent of each other.

This is however not the case on a single-track line. The four train patterns are very interdependent through crossings, overtakings and constraints on acceptable amount of scheduled delay. This means that just a small fraction of the theoretical timetables will be real feasible timetables.

Figure 4 shows that the proportion feasible timetables is very low, or even negligible. The best case, short inter-station distance and fast high-speed trains gives less than 0.05% feasible timetables. This indicates that a single-track line is not easily operated by mixed, regular traffic.

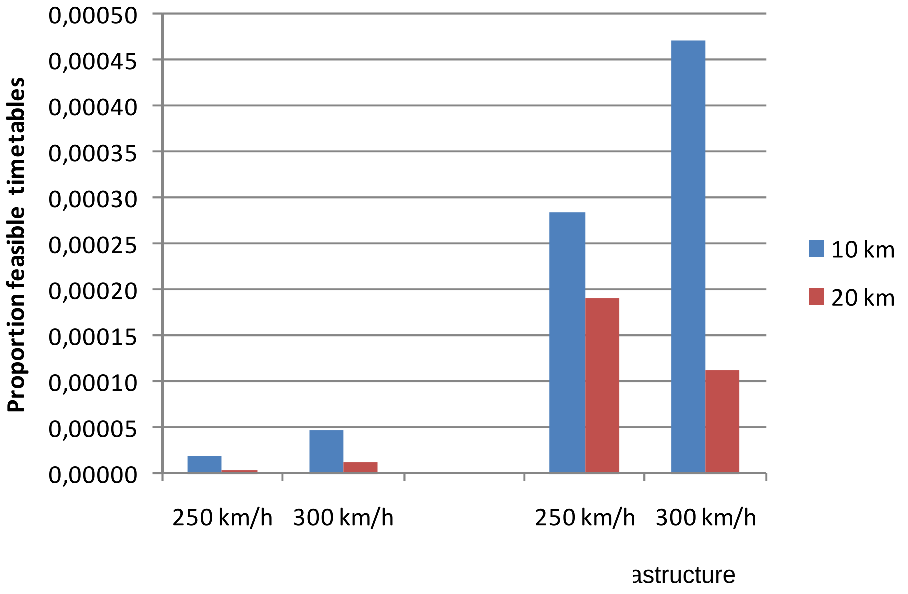


Figure 4 Proportion of feasible timetable variants (timetable flexibility).

Figure 4 also shows a great difference between mean value over all sampled infrastructure designs (100 designs for each inter-loop distance) and the best design. This indicates that timetable flexibility (and capacity) can be gained through if the locations of the crossing loops are adjusted to the intended traffic mix.

Despite very low proportions of feasible timetables, figure 4 shows some interesting results when it comes to the effect of different maximum speeds and inter-loop distances. It is clear that a shorter inter-loop distance increases timetable flexibility. A more remarkable result is that a higher maximum speed for the high-speed trains gives more feasible timetables. This is not intuitive since a higher speed implies more conflicts. One explanation would be that the factors, i.e. the inter-loop distance, the frequency of service and the speed ration between high-speed and regional traffic, interact. Such an interaction might imply that the speed ratio 300/200 is more favourable than 250/200 for inter-station distances that are multiples of 10 km.

High-speed traffic mixed with freight traffic

A mix of high-speed and freight traffic is in some sense easier to schedule than the combination of high-speed and regional traffic. This comes from the fact that the freight traffic is not connected to a regularity requirement, i.e. the freight trains can be scheduled individually. In addition, the acceptance for scheduled delay is higher for freight trains than regional trains. This makes it easier to find feasible train paths and freight traffic can be fitted into patterns of high-speed trains operated at a frequency of 0.5 or 1 train/h/direction.

This is seen in figure 5. The figure shows mean capacity calculated over the timetable variants that are generated through different relative time locations of up- and down bound high-speed trains within all replicates (sampled infrastructure designs).

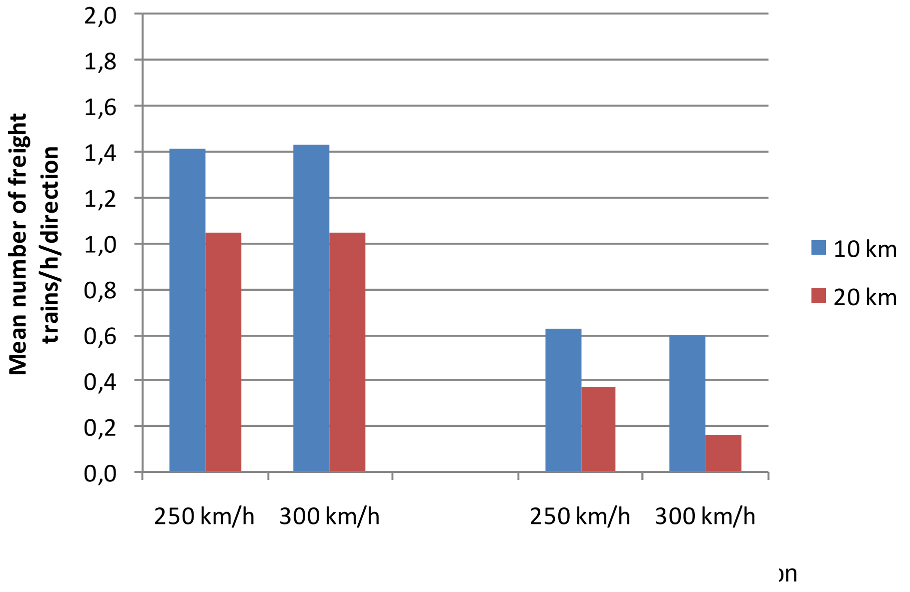


Figure 5 Number of possible freight trains/h/direction. Mean values over all timetable variants within all infrastructure replicates (designs). Average inter-loop distance: 10 km and 20 km.

Figure 5 confirms the conclusions from the analysis of high-speed and regional traffic. If the high-speed traffic has a frequency of 0.5 trains/h/direction there is capacity for 1 freight train/h/direction. The reason why TVEMS manage to schedule these freight trains are mentioned above: higher acceptance for scheduled delay and possibility to schedule bunches, i.e. groups of freight trains (see figure 6 for an example). Note that the difference between 250 and 300 km/h is negligible for the lower high-speed frequency. Even at the higher high-speed frequency, 1 train/h/direction, it is possible to operate freight traffic.

Figure 6 shows a timetable example. The average inter-loop distance is here 10 km and the high-speed traffic is operated at the higher frequency, 1 train/h/direction. It seems that the scheduling algorithm has found efficient solutions where crossing and overtaking are combined at crossing loop 17 and 18. This implies that theses loops have two loop tracks in addition to the main track used by the high-speed train.

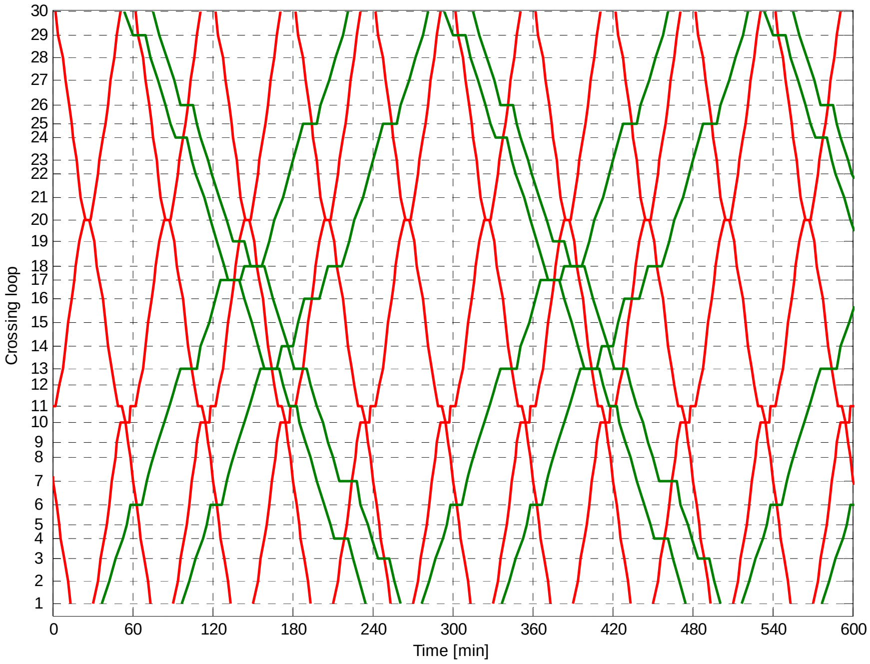


Figure 6 Timetable example with one high-speed train every hour (red) and freight trains fitted in-between these (green). Please note how all three loop tracks on crossing loop 17 and 18 are utilised for capacity efficient combinations of crossing and overtaking.

Design of single-track lines for homogenous high-speed traffic

Crossing areas – where trains cross and suffer knock-on delays

Operation of single-track railway lines is characterized by crossing where trains in different directions meet each other. Two undesirable properties follow on the crossings:

- Increased timetabled run times due to speed restrictions at turnouts.
- Risk of delay propagation and knock-on delays.

The increased run time is caused by speed limitations at turnouts and the fact that at least one of the trains has to brake and let the opposing train pass on the other track. The time loss due to braking is higher for short crossing loops than for extended ones.

All crossings are associated with risk of delay propagation. As soon as two crossing trains are not equally delayed the crossing course will differ from the scheduled situation. Depending on the infrastructure design etc this will lead to more or less delay propagation from one train to the other.

It is possible to limit these undesired effects with thought infrastructure designs. To do so it is necessary to have a somewhat broader perspective. It is not a question of designing individual crossing loops, but rather of designing entire crossing areas with several crossing loops that form a system. This becomes clear when typical distributions for the delay difference for two crossing trains are plotted.

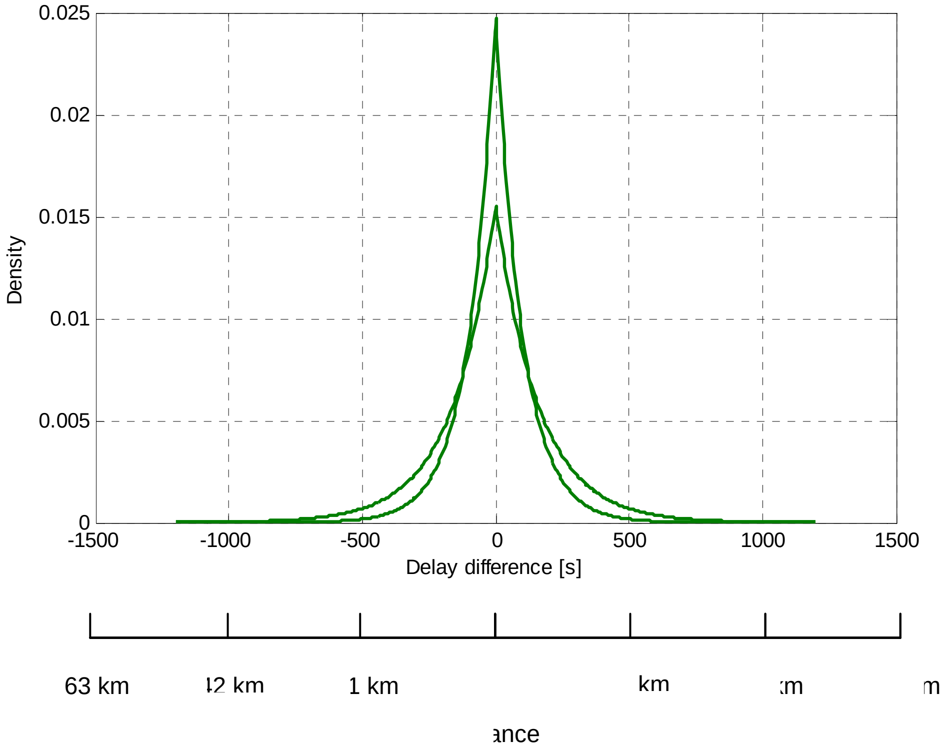


Figure 7 Distributions of delay difference. Lower punctuality results in higher spread in delay difference and a wider crossing area.

Although the crossing is scheduled to take place at delay difference and location zero, the occurrence of delays imply a spread in crossing point. The plotted distributions show the spread in location of the crossing point. The distances corresponds to a train operated at 300 km/h. The figure shows that 40-80 km of double-track is needed to cover all crossings and to avoid delay propagation.

Depending on the actual delay difference, the single-track sections will restrict the crossing more or less. These restrictions are paid for through knock-on delays which mean that delay is propagated from one train to the other.

In this project we use the concept *crossing area* for a line section in which one crossing is to take place. Each crossing area is built up by several crossing loops. The middle crossing loop is called the *scheduled crossing loop* since the crossings are scheduled to take place here. Surrounding loops are denoted *secondary loops* and these are only used in case of great delay differences.

Figure 8 shows three crossing areas. Loop 3, 4 and 5 form one of these. Loop 4 is here the scheduled crossing loop, whereas loop 3 and 5 are secondary loops used for great delayed crossings.

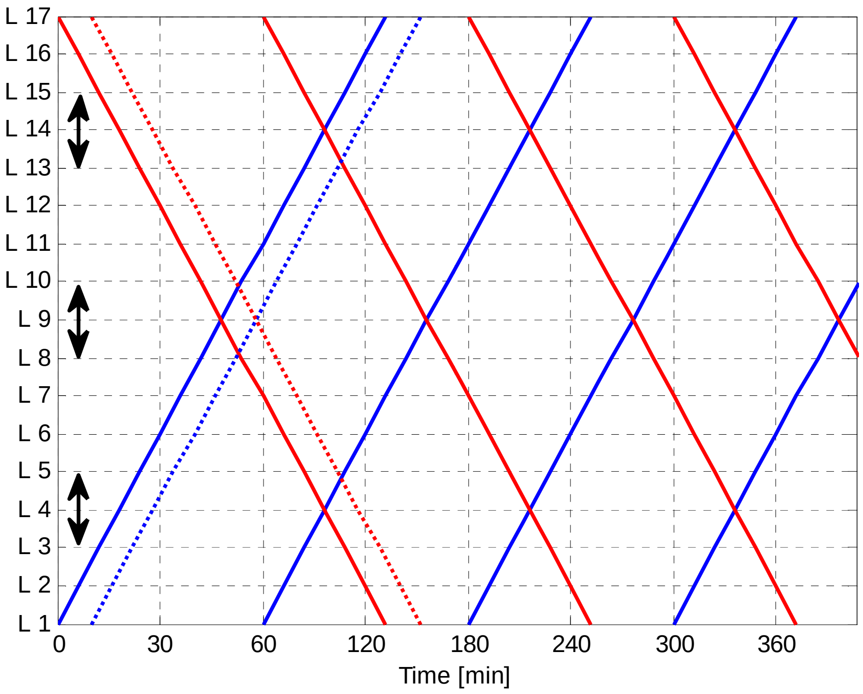


Figure 8 Crossing areas needed for 60-minute traffic. The worst considered delays (10 minutes) are marked with dotted lines. These delays define the crossing area (here three loops in each area).

One aim of this project is to evaluate different infrastructure designs and how they affect operational features. Two infrastructure factors are explicitly addressed; length of scheduled crossing loop and distance between crossing loops. All other infrastructure parameters are kept constant.
Each of the two factors is evaluated on two levels, which means that four different infrastructure designs are tested. Figure 9 shows how these factors combine to the four possible crossing area designs. The first two represent ordinary single-track configurations with just ordinary crossing loops. The second one will probably imply less delay propagation than the first, since the crossing loops are located closer to each other.

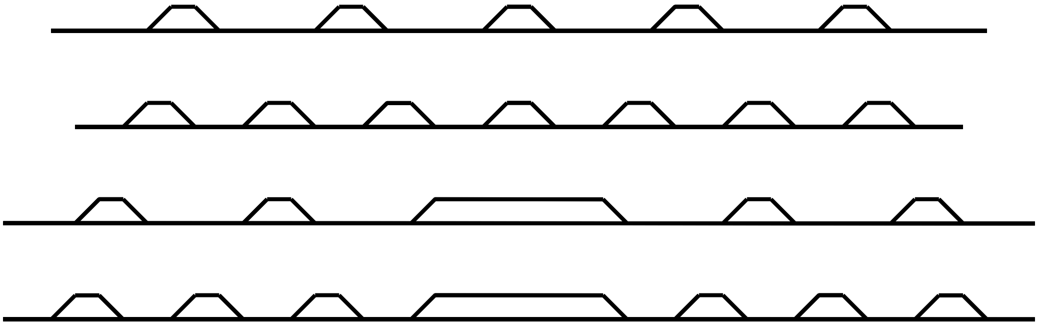


Figure 9 Two factors; length of scheduled crossing loop (middle) and distance between crossing loops, form four different designs for the crossing area.

The two lower crossing areas are designed for time efficient and robust crossings at the extended scheduled crossing loop in the middle.

SAMFOST – a short model description

The crossing situations, where trains travelling in opposite directions meet, are probably the most important part of single-track operation since they cause time losses and delay propagation. The sum of the time loss and the delay propagation is hereafter referred to as crossing time.

In order to examine the influence of crossings a model, named SAMFOST, has been constructed at the Royal Institute of Technology in Stockholm. The model stands on two fundamental assumptions:

1. Two crossing trains are independent before crossing.
2. Different crossing situations are independent of each other. This implies that following trains are always far enough apart not to interfere with each other when they cross a train in the opposite direction.

These assumptions have high validity in non-congested situations, i.e. when the combination of infrastructure configuration, timetabled frequency of services and punctuality is such that following trains only occasionally come so close that independence is broken.

SAMFOST performs a stepwise analysis where each step means that a new set of parameters have to be assigned:

Step 1: The combination of infrastructure and vehicle data together with passenger stop data, give a timetable-free characteristic of the line.

Step 2: The addition of entry delay data gives the full distribution of the delays after a crossing (exit delays).

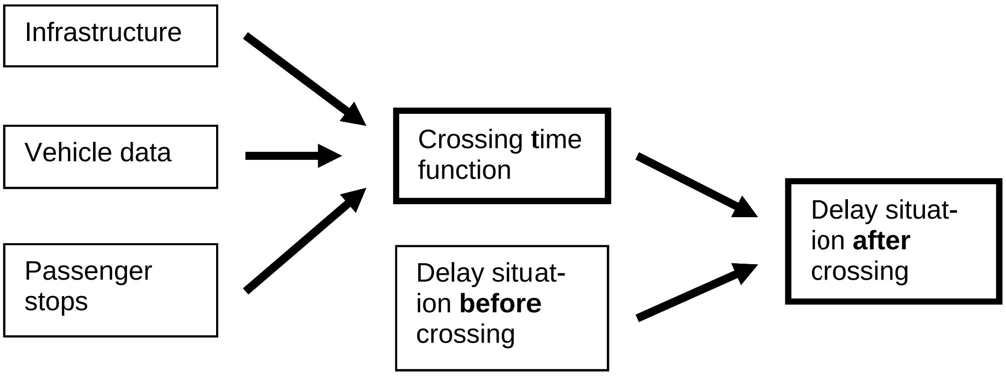


Figure 10 The two model steps in SAMFOST. Bold boxes are output data from the model that may be analyzed further.

The most important advantage of the model is that the first model step is independent of further assumptions about timetable and delays. This makes it possible to define the so-called crossing time function, which is a timetable- and delay-independent description of the infrastructure properties.

The *crossing time* is defined as the extra time needed to perform a crossing on a single-track compared to a double-track where crossings do not imply any extra time consumption. Figure 11 shows an example.

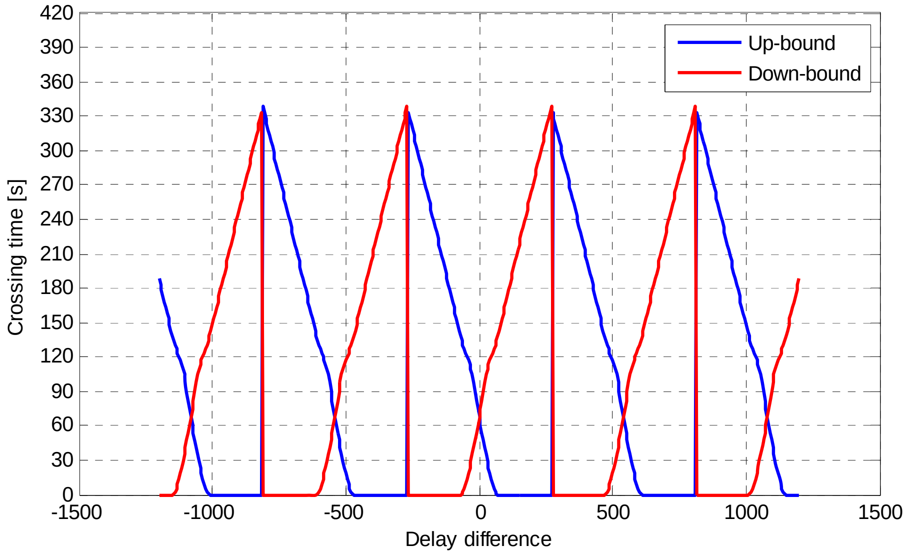


Figure 11 Crossing time function for a line with five, equally spaced, crossing stations.

Without knowing anything about the timetable the crossing time function tells us how the crossing possibilities vary along the studied line. Figure 11 shows the crossing time functions for up- and down bound services on a completely symmetric line with five identical crossing stations.

In figure 11 the crossing is scheduled to take place in the middle, at a delay difference equal to zero. Such a crossing would cost each train ca 75 seconds due to braking at the crossing station. If one of the trains is more delayed than the other this will cause more crossing time to one of the trains and less to the other.

Characteristic features of the crossing time function are their zero-intervals that are interrupted by step intervals where the crossing time increase towards a maximum where it falls back to zero. The maxima corresponds to points exactly between two adjacent crossing stations.

The crossing time function can be used to estimate the time supplement that has to be added in the timetable. However, this estimation will only be correct as long as the trains arrive with a zero delay difference, i.e. when they arrive in the planned time relation to each other.

As soon as the delay difference is non-zero the realized crossing time will take another, higher, value than the planned crossing time. This will be the case as soon as the two crossing trains are not equally delayed. To take this into account it is necessary to combine the crossing time function and a probability density function for the delay difference.

The combination of the crossing time function and the distribution function makes it possible to calculate the delay distribution for a train after a crossing, see figure 12. For different infrastructure designs, giving different crossing time functions, and/or different punctuality levels, resulting in different density functions, it is then possible to compare the effect of different parameters.

An analytical approach like the one used by SAMFOST is very feasible for screening processes where the impact of several factors is to be analyzed. It is very easy to perform calculations for a lot of variants and combinations of infrastructure designs, timetable solutions and delay levels.

SAMFOST has been validated against the simulation tool RailSys. The validation showed that the train runs, i.e. acceleration and braking courses, are modeled in a very similar way in the two models and the differences between SAMFOST and the simulated results (RailSys) turned out to be very small, see Lindfeldt (2010).

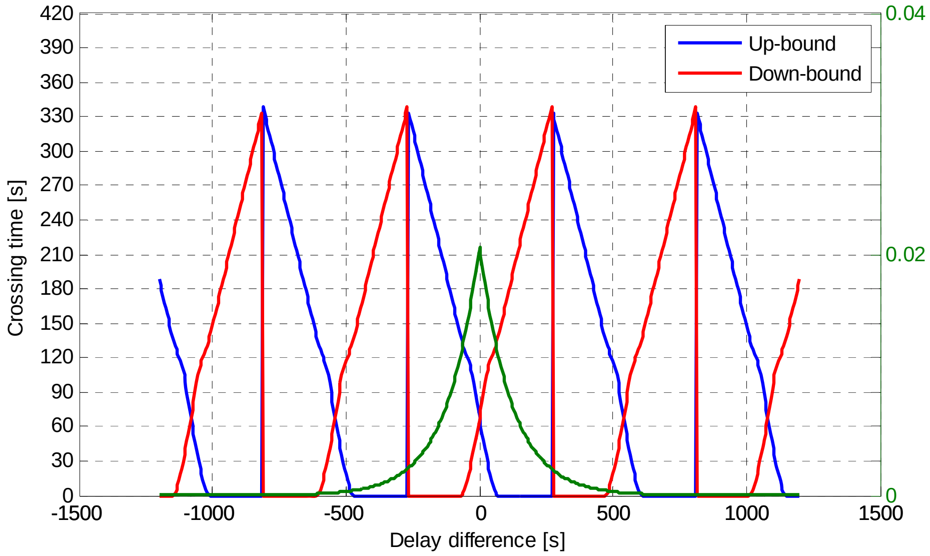


Figure 12 Crossing time and distribution for delay difference.

Measures of performance

Two different measures of performance are chosen for this study. They are used for comparison of different infrastructure designs, vehicle characteristics and delay situations:

- Time supplement needed to restore the delay situation after a crossing.
- Average speed between two adjacent crossings.

Both the increase in run time and the propagation of delays that are caused by crossings have to be handled through time supplements in the timetable. Different infrastructure designs, vehicle characteristics and delay situations require different amount of supplement. It is then reasonable to use the time supplement as measure of performance to compare different combinations of infrastructure design, vehicle and delay situation.

Figure 13 shows an example of two delay distributions, one before crossing (solid) and one after crossing (dashed). The needed time supplement is the time difference at the required delay level (here 95 %) that is needed to restore the delay situation. This is marked by an arrow in the figure.

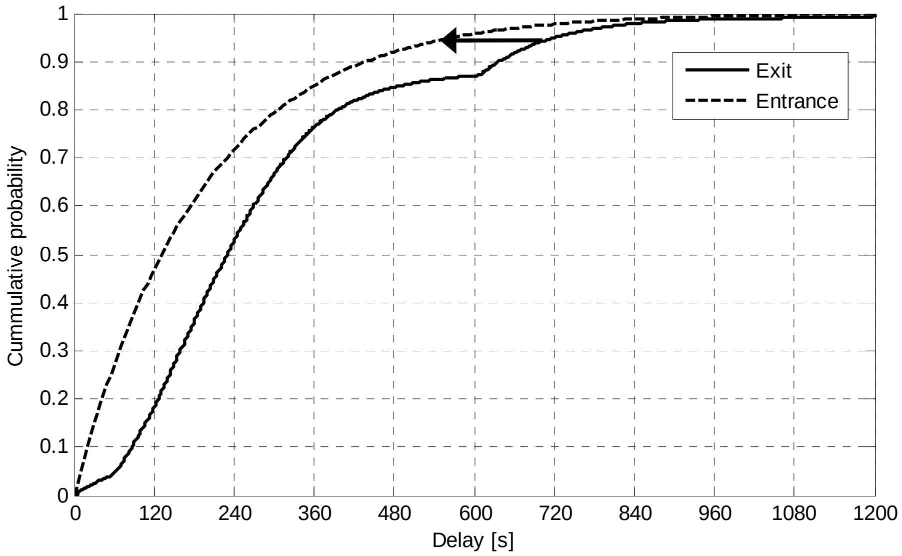


Figure 13 The entry delay distribution (solid) is assumed to be a negative exponential distribution. The crossing means that delay propagates between the trains and so the exit delay distribution is shifted to the right and given another shape.

The supplements also affect the travel times of the trains. High frequent traffic implies more crossings which imply more time supplement and a lower average speed. The average speed is then a good measure of performance since it includes the effect of the frequency of service.

Independent factors

Six independent factors have been chosen for evaluation:

- Length of scheduled crossing loop
- Distance between crossing loops
- Passenger stop at scheduled crossing loop
- Maximum speed / vehicle characteristics
- Delay level
- Frequency of service

The first four factors are referred to as technical factors and the last two as operational factors. The technical factors include the infrastructure design and the vehicle characteristics that directly affect the crossing time function. The operational factors are introduced in a second step to determine the amount of time supplement needed in the timetable.

Each factor appears at two levels in the evaluation and this result in $2^6 = 64$ evaluated variants. This gives a screening of the expected performance for different combinations of infrastructure design, vehicle and operational conditions.

Length of scheduled crossing loop

Normally crossing loops are designed for crossings where the first arriving train is stopped on the side track to wait for the opposing train. This design is feasible when the run times of the trains is not so high valued.

For high speed lines there are extraordinary requirements for short travel times and so the infrastructure has to be designed for time efficient crossings. The length of the crossing loops is of great importance for the crossing time. A longer loop means less delay propagation since it is possible to perform time efficient crossings within an interval of delay difference, which implies less delay propagation.

Two different lengths for the scheduled crossing loop have been chosen to analyse this effect:

- 1 500 m
- 20 000 m

The short loop, with a track length of 1 500 m, is an ordinary crossing loop that has been extended to give time efficient crossings for small delay differences (punctual crossings). The length is chosen to be slightly longer than the braking distance for a train operated in 160 km/h. This means that an arriving train can pass the turnout at 160 km/h and then do the remaining braking on the side track.

The longer loop, with a track length of 20 000 m, is a partial double-track. It is long enough to allow crossings where the trains do not interact through the signalling system. This means that also somewhat less punctual crossings, with moderate delay differences, are accomplished at a minimum of crossing time. The length is chosen to cover most of the probability mass for the applied delay situations.

Previous works has examined both trapezoid and rhomboid designs of crossing loops. This evaluation only includes trapezoid designs. The main reason for this is that the utilisation rate of a partial double-track cannot be guaranteed, i.e. there will be situations without crossing on the partial double-track. In these cases a rhomboid design would cause unnecessary scheduled delay to non-crossing trains.

Another reason for excluding rhomboid designs is the occurrence of passenger stop on partial double-tracks. Previous studies have shown that such stops should be located at braking distance from the turnout at the end of the crossing loop. This location of the stop means that the effect of turnout speed restriction is eliminated.

Distance between crossing loops

Crossings are scheduled to take place at a specific crossing loop. However, in cases with high delay differences, the crossing location is often changed to an adjacent crossing loop. This means that the inter-loop distance affects the crossing time since the waiting time for the first arriving train will be dependent on the distance to the next crossing loop.

Two different inter-loop distances have been chosen for analysis of this effect:

- 20 000 m

- 10 000 m

This distance is the length of the single-track section between turnouts at adjacent crossing loops and it is of greater importance when large delays are frequent.

The two infrastructure factors, loop length and distance between loops, are modelled on two different levels each. This results in the four designs of the crossing area shown in figure 14.

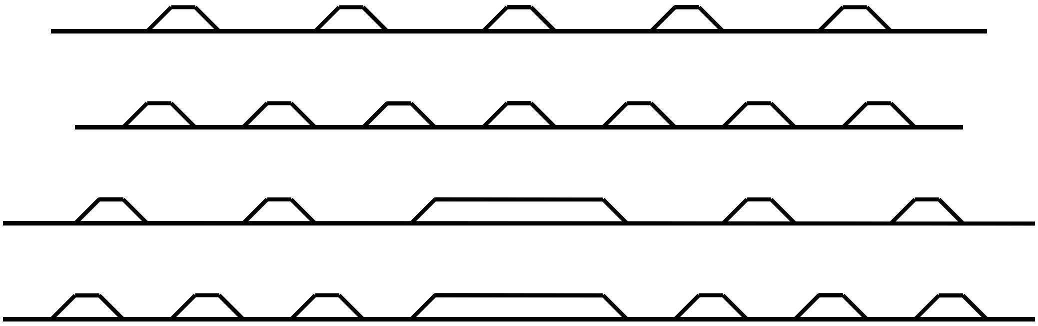


Figure 14 The four evaluated designs of crossing areas.

Passenger stop at scheduled crossing loop

Passenger stops are preferable located at crossing loops since this widen the time window within which crossings without delay propagation can be accomplished. Scheduled dwell on single-track sections have the opposite effect since they prolong the run time between two crossing loops which in time increases the risk and the amount of delay propagated between opposing trains.

Using 160 km/h as speed restriction in turnouts, the optimal location of the passenger stop is at braking distance from the restrictive point, i.e. at the end of an ordinary crossing loop of 1 500 m length and approximately 1 500 m from either end of an extended loop.

Also the passenger stop factor was assigned on two levels:

- No passenger stop within the crossing area.
- Passenger stop (120 s) at the scheduled crossing loop.

A passenger stop means that time supplement for braking, dwell and acceleration has to be included in the timetable. This has to be explicitly shown in the evaluation.

Maximum speed / vehicle characteristics

Two types of train sets were used in this study to investigate the influence of two different top speeds on the line. One existing modern high speed train, ICE3- Velaro (see Figure 15) for speeds of up to 300 km/h and a generic EMU with a top speed of 250 km/h. The basic train data are shown in Table 2.

Figure 15. Siemens Velaro (AVE S-103) high speed train 1+2 and 2+2 seating. Standard body width. Total capacity 536 seats (8 cars). Commercial speed 350 km/h
Source: Wikimedia. http://commons.wikimedia.org/wiki/File:RENFE_AVE_S-103_15.jpg

Table 2. Basic train data for ICE3 Velaro and a Generic EMU.

	ICE3 Velaro (EMU)	Generic EMU
Speed in study	300 km/h (top speed 350 km/h)	250 km/h
Max power	8800 kW	9600 kW
Starting effort	280 kN	353 kN
Starting acceleration	0.60 m/s^2	0.70 m/s^2
Average deceleration	0.75 m/s^2 (0.98 m/s^2 possible)	0.85 m/s^2
Acceleration time	300 s (0-300 km/h)	180 s (0-250 km/h)
Acceleration distance	17000 m (0-300 km/h)	8000 m (0-250 km/h)
Mass	440 t	420 t
Length	200 m	200 m

It is worth noting that the ICE3-Velaro is designed to run up to 350 km/h and has in this study an excess of power which can be used to cope with grades and maintain the desired operational speed. The performance charactersistics of the ICE3 train is not changed for this study and the tractive effort together with running resistance on level track is shown in Figure 16.

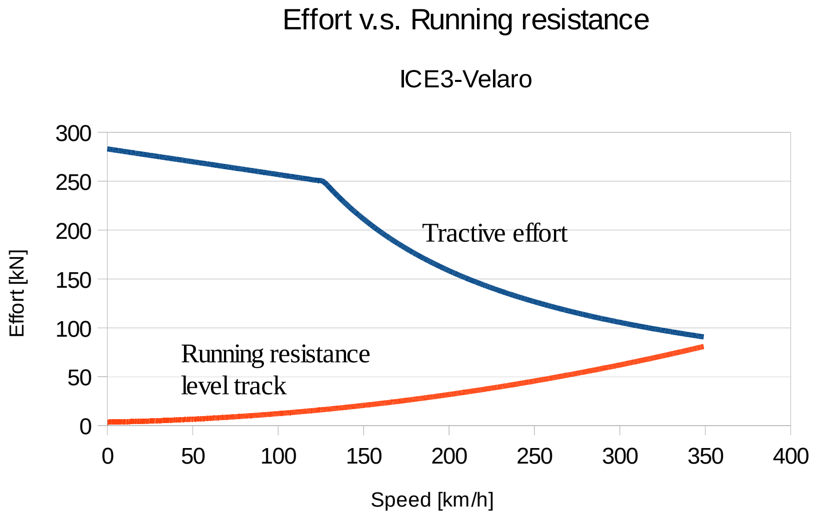


Figure 16. Tractive effort versus running resistance on level track for ICE3- Velaro.

Since very long grades can have an influence on average speed and delays, Figure 17 shows an example of the impact of grades on top speed for ICE3-Velaro. The Generic EMU has about the same performance as the ICE3 Velaro up to 250 km/h. However, in this study the line is assumed to be level.

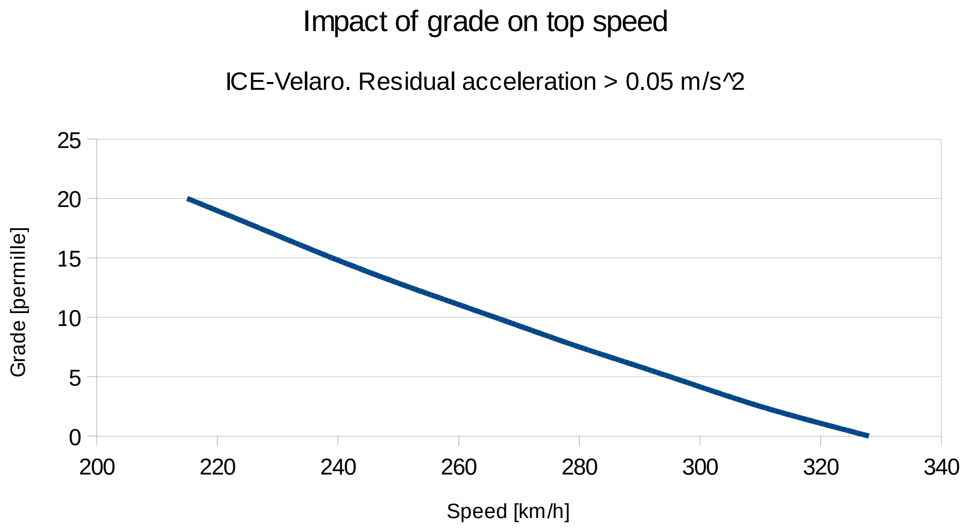


Figure 17. Impact of grade on top speed for ICE3- Velaro.

Delay level

The entry delays, i.e. delays before crossing take place, are very important for the delay propagation on single-track lines. A delay situation is defined by the entry delay distributions for the crossing trains. For simplicity we assume up- and down bound trains to have the same delay level, i.e. to have the same entry delay distribution.

The negative exponential distribution is commonly used for railway operation analysis. The following delay levels have been chosen based on data presented in the chapter "Acceptance criteria for punctuality":

- Mean entrance delay: 100 s (1,7 min)
- Mean entrance delay: 160 s (2,7 min)

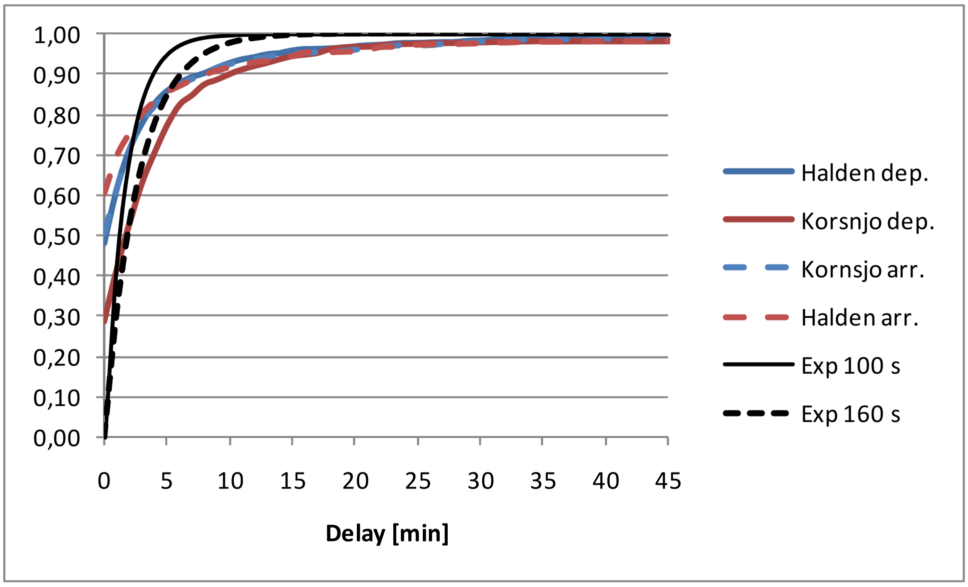


Figure 18 Entry delay distributions. Two different distributions are used: Exp 100 and Exp 160. These have different properties than the empirical distributions from existing Norwegian operation.

Figure 18 shows these two exponential distributions together with distributions for the existing Norwegian operation. The Exp 160-distribution has a mean delay of 160 s (2,7 min). This is at the same level as the current traffic. Note that the punctuality on five-minute level is 85 % both for Exp 160 and three of the existing distributions.

One major difference however, is that the Exp 160-distribution is worse for lower delays than five minutes, i.e. delays within 0-5 minutes is more frequent than in the real operation. The opposite is true for great delays. Here the existing operation shows higher frequencies than the Exp 160.

These differences must be kept in mind. The assumed Exp 160-distribution gives a more ideal situation with less great delays, i.e. severe infrastructure and vehicle failures etc. It is usually these great delays that cause problems on single-tracks, whereas the small delays can be treated by small time supplements in the timetable.

The other distribution used within this project, Exp 100, corresponds to 95% punctuality. This distribution is even more feasible for single-track operation since great delays are less frequent.

Frequency of service

The frequency of service does not affect the crossing time and the supplement needed to manage each crossing situation. However, a higher frequency of service implies that crossings occur more often which means that the total time supplement increases. For this reason it is also useful to analyse the effect of two different frequencies of service:

- One train per hour (60 minute interval).
- Two trains per hour (30 minute interval).

A frequency of one train per hour means that every train has to cross another train every 30 minutes, whereas a frequency of 30 minutes implies a crossing every 15 minutes.

Constants and other assumptions

Several factors have been modelled as constant, although a more thorough screening would include also some of them. Some of the constants are:

Loop type. Only trapezoid loops are modelled, see section about loop length.

Turnout speed. All turnouts are modelled with a speed restriction of 160 km/h. This is reasonable for extended crossing loops (partial double tracks) in order to limit the time loss for side track train routes. For crossing loops of normal length this speed could be over-standard, at least for secondary loops that are not used as scheduled crossing loops. In practice, turnouts located close to a regular stop do not necessarily need to be given a higher standard than 100 km/h since this speed restriction does not affect trains accelerating from a stand still.

Length of secondary loops. All secondary loops are given the same properties, i.e. 1 500 m track length. This gives a high standard for loops that are not, initially, planned to be used as scheduled crossing loops. This standard is very useful both for rush hour trains and in case of major timetable variations.

Interlocking time. The time to release a train route and set a new is assumed to be 20 s. This time includes transmission times and switching time for the turnouts.

Signalling system. The modelled signalling system corresponds to ERTMS level 2 (or 3), which means that driving permissions are continuously updated.

Track disposition. The dispatching principle used by the model means that the total crossing time is minimized. This principle is used both for choice of crossing loop and for track utilisation. It means that the first arriving train uses the side track so that the total crossing time is minimized. It is hereby assumed that the dispatching system has the ability to recognise which of two crossing trains that will first arrive to its entrance turnout.

Gradient. The entire line is assumed to be horizontal.

Braking rate. The braking rate is assumed to be constant, see the section about vehicles.

Maximum speed. The maximum line speed is constant and equal to the maximum speed of the analyzed vehicle.

Results

Four technical factors have been evaluated on two levels:

- Length of scheduled crossing loop
- Distance between crossing loops
- Passenger stop at scheduled crossing loop
- Maximum speed / vehicle characteristics

These four factors result in 16 crossing time functions, see appendix 2, which show how the infrastructure design and the vehicles interact during crossing courses. The four combinations of loop length and inter-loop distance are listed in Table 3.

	Loop length [m]	**Inter-loop dist.[m]**
1	1 500	20 000
2	1 500	10 000
3	20 000	20 000
4	20 000	10 000

Table 3 Analyzed combinations.

Crossing supplements

The crossing time functions for these 16 combinations were in turn combined with the two delay levels: 100 seconds mean delay / 95 % punctuality and 160 seconds mean delay / 85 % punctuality. This resulted in 32 values of time supplement needed for every crossing situation presented in figure 19 and 20.

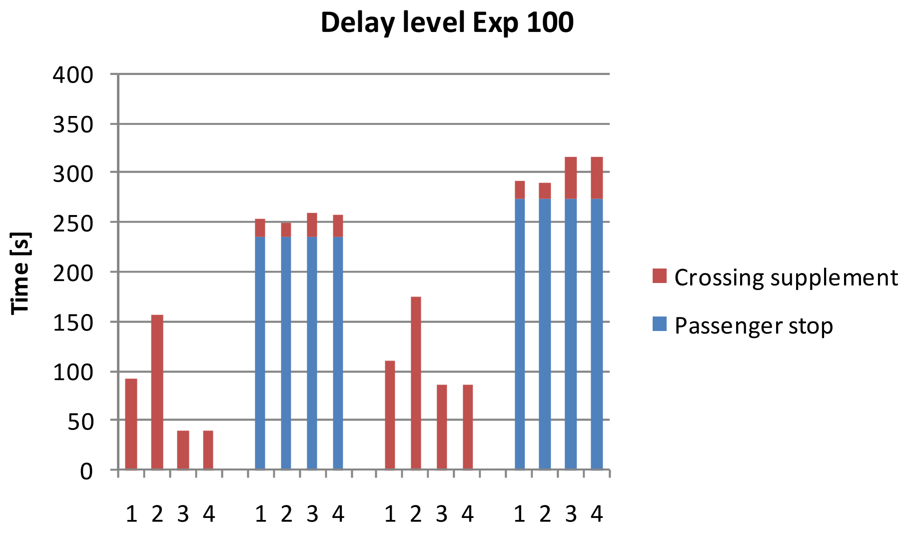


Figure 19 Required time supplements when entry delays are Exp(100)-distributed. Loop length and inter-loop distance according to table 3.

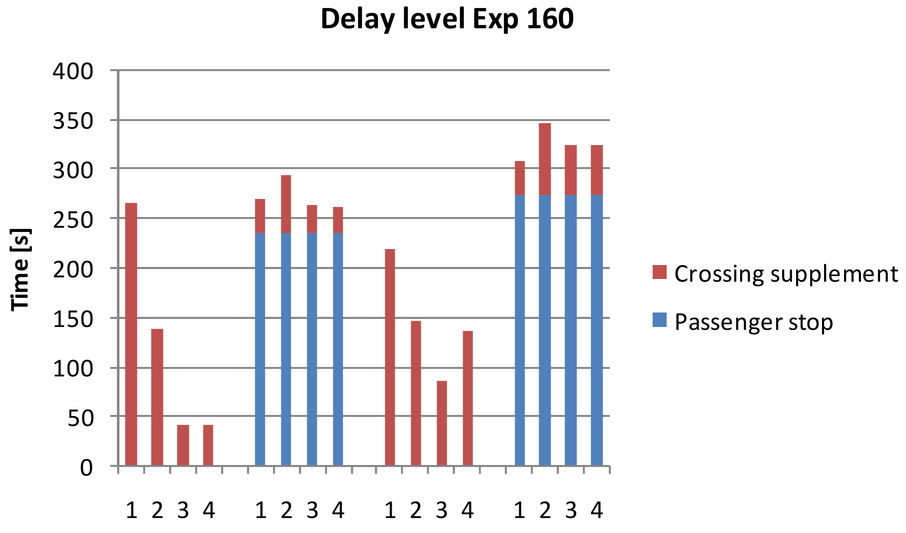


Figure 20 Required time supplements when entry delays are Exp(160)-distributed. Loop length and inter-loop distance according to table 3.

The only difference between figure 19 and 20 is the delay level and acceptance level for punctuality. It can be seen that a higher punctuality most often, but not always, requires less time supplement.

Two main groups are seen in each figure: 250 km/h are the leftmost eight bars and 300 km/h are the rightmost eight bars. In most combinations a higher maximum speed requires more time supplement. The difference is higher when the crossing is combined with a passenger stop, since the deceleration, dwell and acceleration then has to be accomplished for both trains in all crossing situations, independent of the actual delay difference.

The passenger stop, however means that the extra supplement needed to reduce delay propagation (crossing supplement) is limited. Please note that this effect depends on the delay level. Higher punctuality (Exp(100)) requires less time supplement than lower punctuality. Collocation of crossing and passenger stop is therefore beneficial since almost no crossing supplement is needed since the dwell time makes it possible to perform time efficient crossings during a rather wide interval of delay difference.

It is also important to notice that a passenger stop makes the crossing time supplement quite independent of loop length and inter-loop distance. This means that a crossing loop of 1 500 m length is enough when the crossing is combined with a passenger stop. It is also unnecessary to reduce the inter-loop distance from 20 000 to 10 000 m in this case.

The situation is somewhat different when a scheduled crossing is planned to a loop without passenger stop (combination 1-4 and 9-12). In this case the most efficient way to reduce the crossing supplement is to extend the scheduled crossing loop into a partial double-track. This is seen in the lower bars 3, 4, 11 and12.

All the results mentioned above are reasonable. There is however one non-intuitive result. This is seen when comparing bar 1 with 2 (or 9 with 10). These bars show that a shortened inter-loop distance, from 20 000 m to 10 000 m, results in more crossing supplement. The reason for this is that the inter-loop distance affects the shape of the exit delay distribution. This shape change is concentrated to great delays, i.e. to the area where the punctuality is taken (85 or 95 %). For details please have a look in appendix 3, which shows the entry and exit delay distributions for all 32 cases.

The conclusions regarding inter-loop distance is that a change from 20 000 m to 10 000 m does not affect the need for crossing supplement very much. This result implies that great delays are not more frequent than the tested distributions. Previous work, Lindfeldt 2007 and Lindfeldt 2010, has shown that the inter-station distance is of much greater importance when punctuality is low.

Average speeds

The crossing supplements and the supplements for passenger stops affect the average speed of the trains. One way to compare the different infrastructure and vehicle combinations is to calculate the achievable average speed for each case. Such calculations require that the frequency of service is known since this frequency affects the number of crossings to be accomplished during a given time period.

Figure 21 and 22 show the achievable average speeds for two levels of frequency of service: one train/hour and two trains/hour.

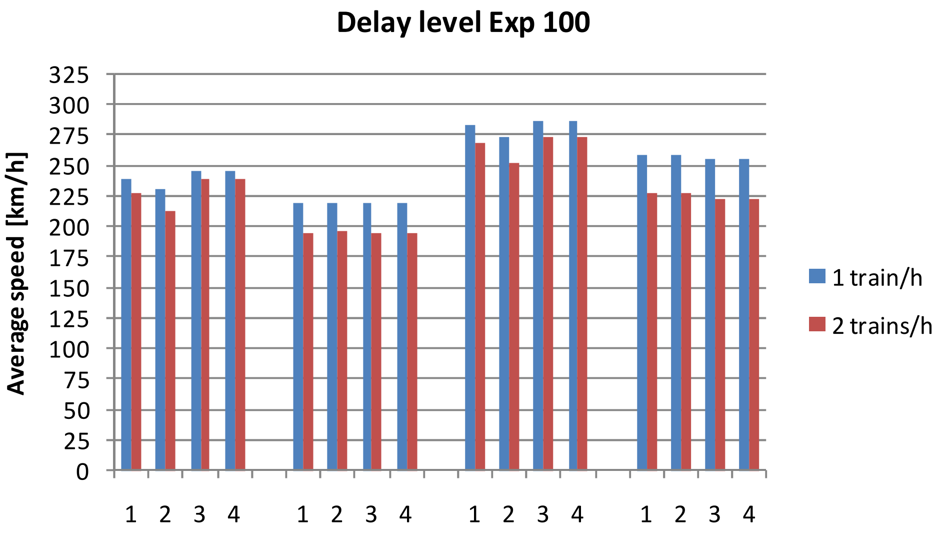


Figure 21 Achievable average speed when entry delays are Exp(100)-distributed. Loop length and inter-loop distance according to table 3.

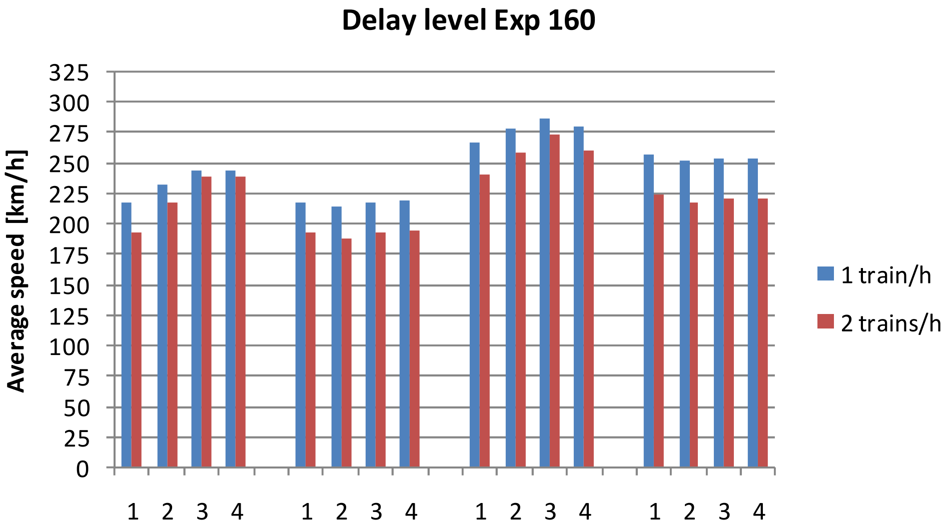


Figure 22 Achievable average speed when entry delays are Exp(160)-distributed. Loop length and inter-loop distance according to table 3.

A brief comparison of the two delay levels shows that the average speed is not very sensitive to the delay level. Combination 3, 4, 11 and 12 give the greatest differences: 5- 6 km/h with two trains/h-operation. These combinations are extended crossing loops (20 000 m) without passenger stop. They are also ones that are least sensitive to the frequency of service, i.e. the average speed is independent of the how oft crossings are to be accomplished.

Passenger stops imply significant drop in average speed, 10-37 km/h depending on infrastructure design, maximum speed and delay level. The faster trains have a greater speed drop (18-37 km/h) than the slower ones (10-29 km/h) for a passenger stop.

A real railway line will be composed by several different crossing areas (combinations) put together after each other. This means that the realized average speed over the entire line will be a weighted mean value of the average speed on each crossing area.

Please note that the presented speeds do not include ordinary time supplements of 3-5 % of the run time for recovery from primary delays. These supplements will reduce the average speed somewhat further, but can be approximately compensated for if the average (top-) speed is increased.

Conclusions

This report has two main objectives: to analyze capacity for mixed traffic on single-track lines and to make a deeper study of single-track lines designed for homogeneous high-speed traffic. The second step is of interest since capacity and quality can be gained on single-track lines if traffic is homogeneous and the infrastructure is designed for just homogeneous traffic with a limited number of well defined crossing areas.

Capacity for mixed traffic

Mixed traffic is a challenge already on double-track lines where faster trains catches up with slower ones. For single-track lines this phenomenon is combined with crossings. This means that both crossings and overtakings must be handled. The catch-up effect also results in timetable wedges which in turn makes it difficult to find feasible timetable solutions for crossings. It is simply too time consuming to wait for a train to wait for two crossing trains at the same crossing loop.

Together these operational characteristics limit and restrict capacity and timetable flexibility on single-track lines operated with mixed traffic. Based on the performed analyses we want to make the following recommendations:

High-speed and regional traffic

- The frequency of service is the most important and constraining factor.
- Timetable flexibility is generally very low for this kind of traffic. It is difficult to change or develop the traffic
- A frequency of 0.5 high-speed trains/h/direction and 0.5 regional trains/h/direction will work on a normal[4] single-track line.
- A combination of 1 train/h/direction for one of the train types and 0.5 trains/h/direction for the other will probably work on an adjusted[5] single-track

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[4] A normal line is not constructed with a special timetable in mind. It is rather adjusted to manage timetable changes and development.

line. We did not manage to verify this since more work has to be put on how to adjust the infrastructure.

- The inter-loop distances should be chosen with intended speed ratios and frequencies of service in mind.

High-speed and freight traffic

- The frequency of service is the most important and constraining factor.
- Timetable flexibility is generally low also for this kind of traffic.
- A frequency of 0.5 high-speed trains/h/direction and 1 freight train/h/direction will work on a normal[6] single-track line.
- A frequency of 1 high-speed train/h/direction and 0.5 freight trains/h/direction will also work on a normal single-track line with average inter-loop distance 10 km.

Homogeneous high-speed traffic

- The homogeneity increases the capacity since no catch-up effects occur. It is also easier to adjust the infrastructure for time efficient crossings since the number of crossing areas is lower.
- A frequency of 1 high-speed trains/h/direction will work on a normal single-track line.
- A frequency of 2 high-speed trains/h/direction will probably work on an adjusted single-track line. We did not manage to verify this since more work has to be put on how to adjust the infrastructure, but the special analysis of homogeneous high-speed traffic supports this supposition.

Design of single-track lines for homogenous high-speed traffic

The analysis presented in this report is an analysis of a high-performing system. Several factors were set to values not used within ordinary railway operation:

- Each crossing loop has turnouts for 160 km/h and the loop length is adjusted to this speed. This makes it possible to perform time efficient crossings at every crossing loop. This is useful in delayed situations and makes it possible to alter the timetable, add extra trains etc. In practice, turnouts located close to a regular stop do not necessarily need to be given a higher standard than 100 km/h since this speed restriction does not affect trains accelerating from a stand still.
- Ordinary as well as extended inter-loop distances have been analyzed. It is seen that the difference between 20 000 and 10 000 m inter-loop distance is limited due to the high punctuality assumed. However, a shorter inter-loop distance

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[5] An adjusted line is a line where certain parts are constructed for time efficient crossings and overtakings, e.g. partial double-tracks, loops with four tracks for combined crossings and overtakings etc.

[6] A normal line is not constructed with a special timetable in mind. It is rather adjusted to manage timetable changes and development.

would contribute to timetable flexibility, i.e. give more feasible timetable solutions.

- Like in previous studies passenger stops turned out to be very appropriate for crossings. A crossing loop of 1 500 m length with 160-turnouts (at the farther end) showed to be a very feasible design of a crossing loop. The loop does not need to be extended since delay propagation is limited at the assumed delay levels.
- It might not be possible to collocate crossing and passenger stop. These situations call for other infrastructure solutions. An extended crossing loop is a feasible alternative in this case. An extension to 20 000 m loop length reduces the time supplement by 55-80 seconds per train.
- It is possible to design a single-track line for a maximum speed of 300 km/h. A shift from a top speed of 250 km/h to 300 km/h means that the average speed is increased by 35-50 km/h for crossing areas without passenger stop and 30-40 km/h for crossing areas with passenger stop. This shows that the infrastructure can be designed so that most of the speed increase can be utilised.
- All results and conclusions presume a low delay level. More time supplement would be needed if the delay situation gets worse with more great delays.
- Punctuality is a simplified measure of the operational performance. It is easy to apply and use for calculation of supplements needed for restoration after a crossing. However, delayed crossings alter the shape of the delay distribution in such a way that a single point on the distribution (punctuality) is not always completely representative for the delay situation.

Recommendations and further studies

The recommendation is to perform further studies in the following fields:

- Exact length of extended loops for scheduled crossings without passenger stop.
- Design of crossing areas close to connecting line sections with worse punctuality. Additional design features and time supplements might be needed in these crossing areas.
- Timetable flexibility and capacity. Analysis of interaction effects between adjacent crossing areas.
- Detailed studies, e.g. with simulation, of robustness and recovery.
- The impact of gradients.
- Analysis of impact of bad weather conditions. low friction conditions.
- Optimal train performance, top speed, acceleration etc...
- Power need and energy consumption.
- Possibilities for eco driving and optimal train control, thus increasing punctuality.

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Lindfeldt, O., (2010). *Railway operation analysis – Evaluation of quality, infrastructure and timetable on single and double-track lines with analytical models and simulation*, Thesis (PhD), Stockholm.

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Appendix 1

Table A1. Basic train data for ICE3 Velaro, Generic EMU and Generic freight train of uniform consist on level track.

	ICE3 Velaro E	Regional Generic EMU	Freight train Covered type Wagons
Speed in study	Utilized top speed in study: 250 / 300 km/h (maximum top speed 350 km/h)	200 km/h	140 km/h
Max power	8800 kW	9600 kW	5600 kW
Starting effort	280 kN	353 kN	300 kN
Starting acceleration Level track	0.60 m/s^2	0.70 m/s^2	0.49 m/s^2
Average deceleration	0.8 m/s^2	0.8 m/s^2	0.80 m/s^2
Acceleration time to top speed	300 s (0-300 km/h)	125s (0-200 km/h)	115 s (0-140 km/h)
Acceleration distance to top speed	17000 m (0-300 km/h)	4100 m (0-200 km/h)	2700 m (0-140 km/h)
Mass	440 t	420 t	564 t
Length	200 m	200 m	157 m

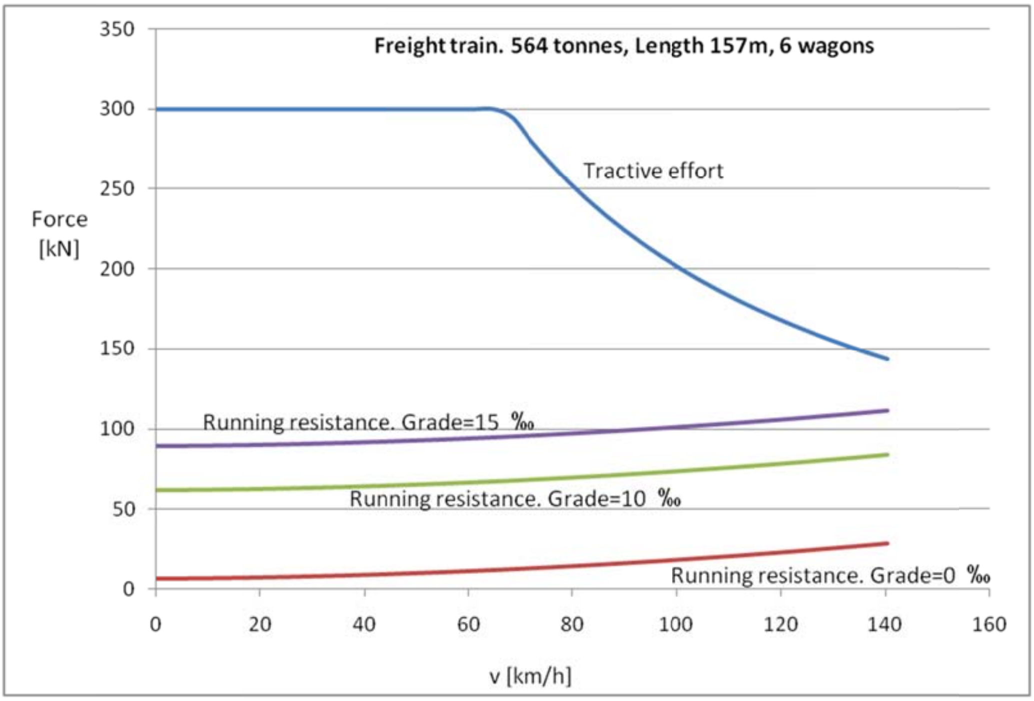


Figure 23. Tractive effort and running resistance for the generic freight train in grades of 0, 10 and 15 permille. The generic freight train in this study represents a "light freight train" designed to run at 140 km/h. Length 157 m, mass 564 tonnes, 6 wagons.

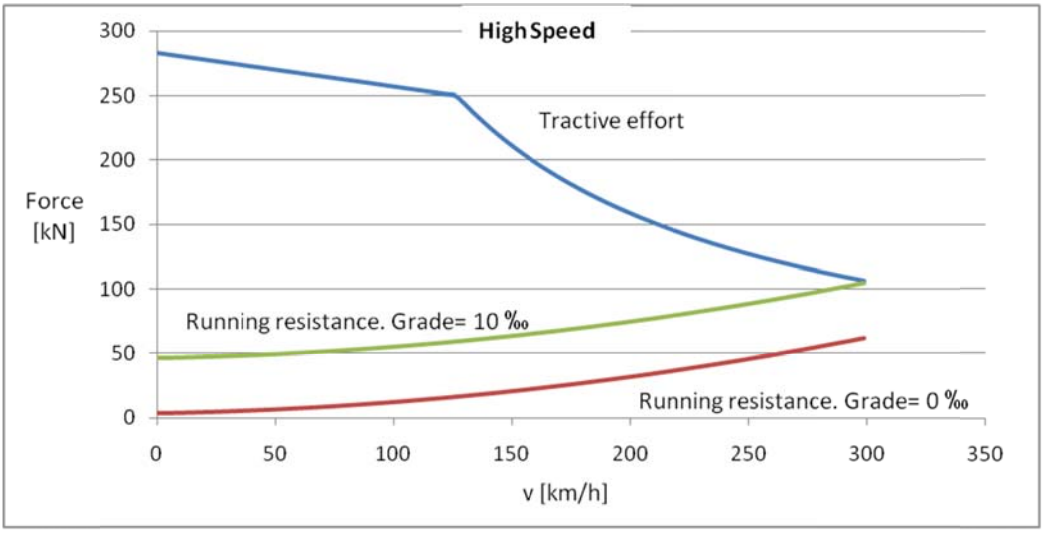


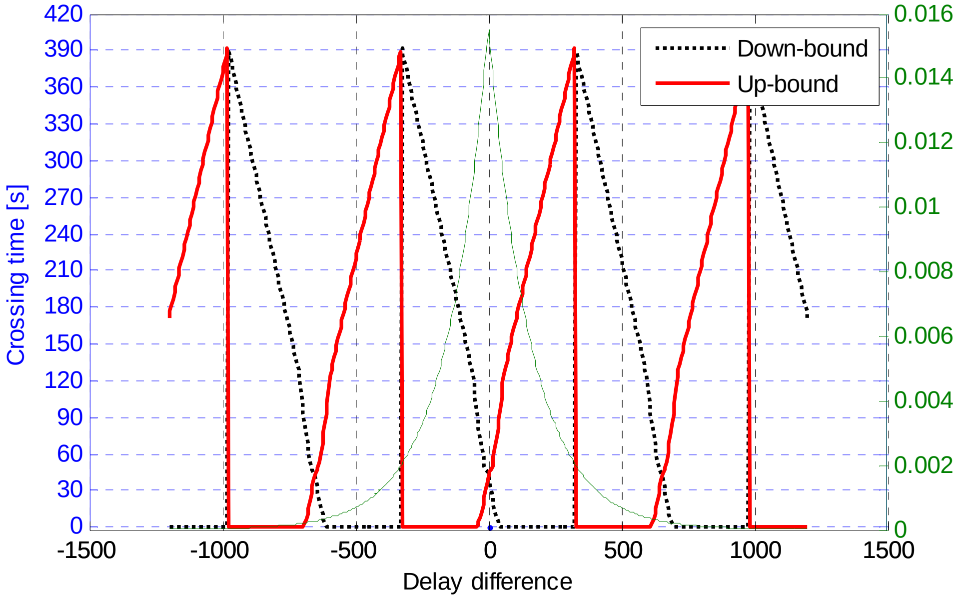
Figure 24. Tractive effort and runnnig resistance for ICE3 Velaro- E in grades of 0 and 10 permille up to 300 km/h (ICE3 Velaro can run up to a speed of 350 km/h).

The performance of the generic EMU is similar to that of ICE3- Velaro, up to a speed of approx. 250 km/h

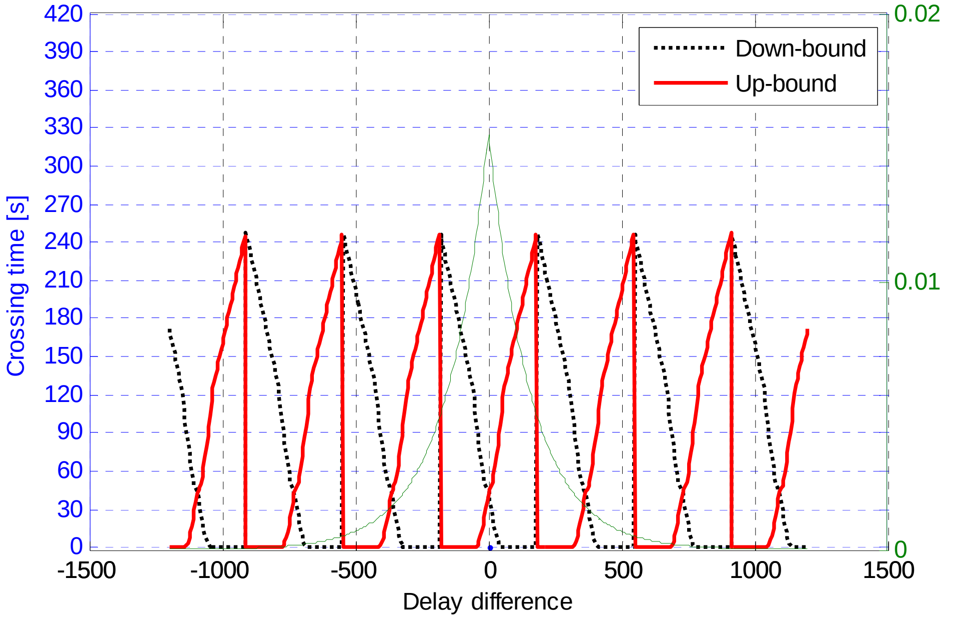
Appendix 2: Crossing time functions

Each crossing time function is completed with the distribution for delay difference that corresponds to entry delay distributions Exp(160s), i.e. the worse delay situation.

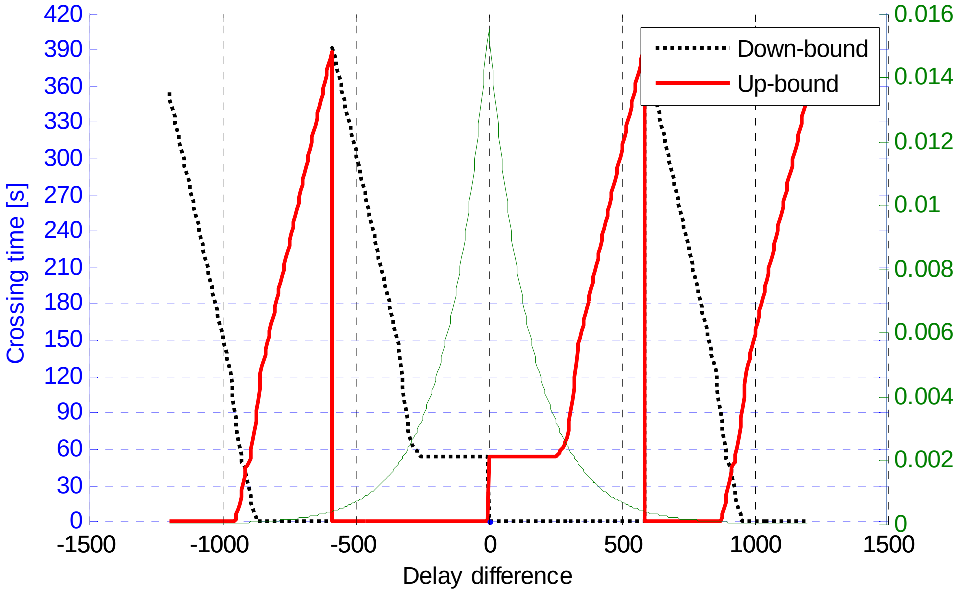
1
Max speed: 250 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 20 000 m



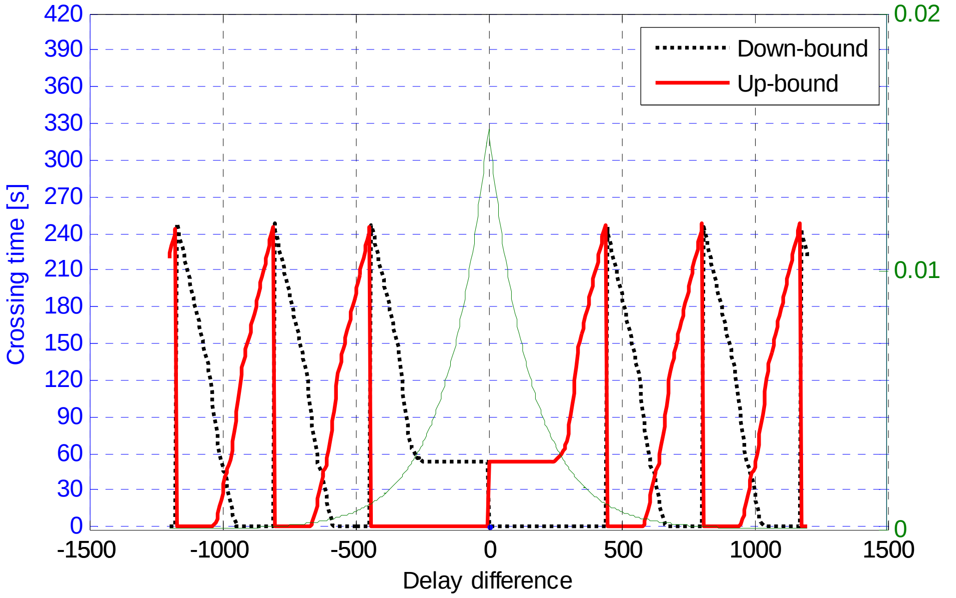
2
Max speed: 250 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 10 000 m

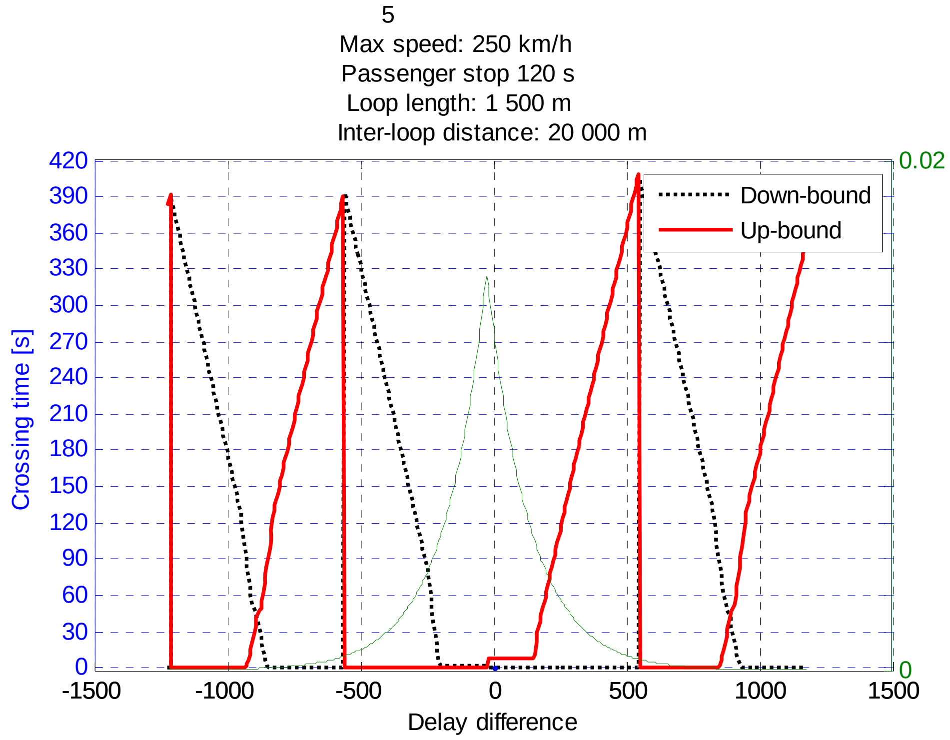


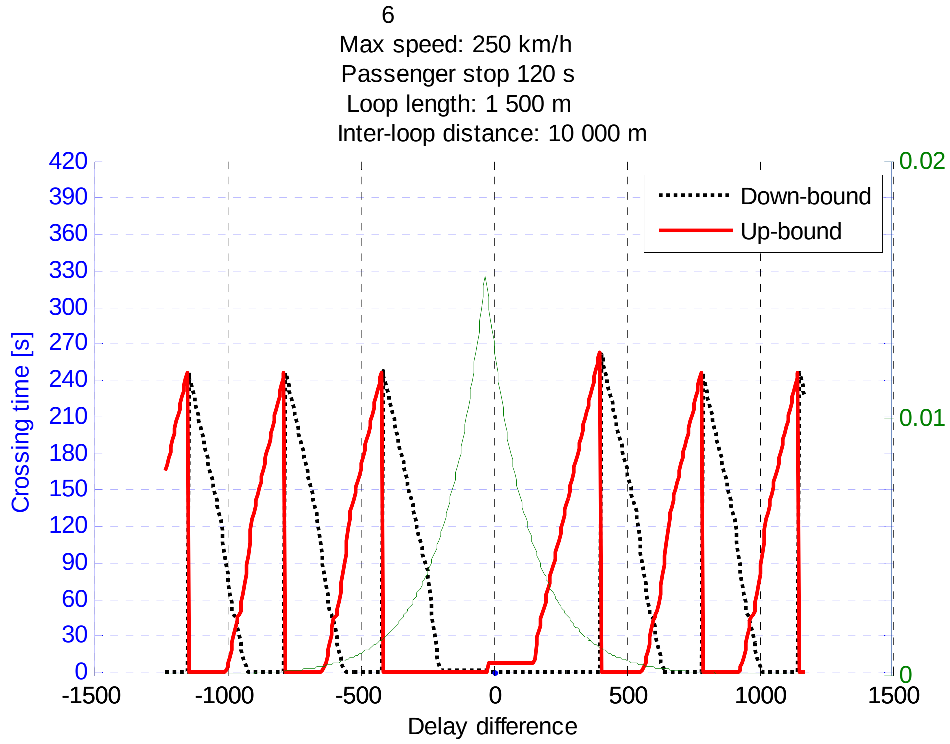
3
Max speed: 250 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 20 000 m



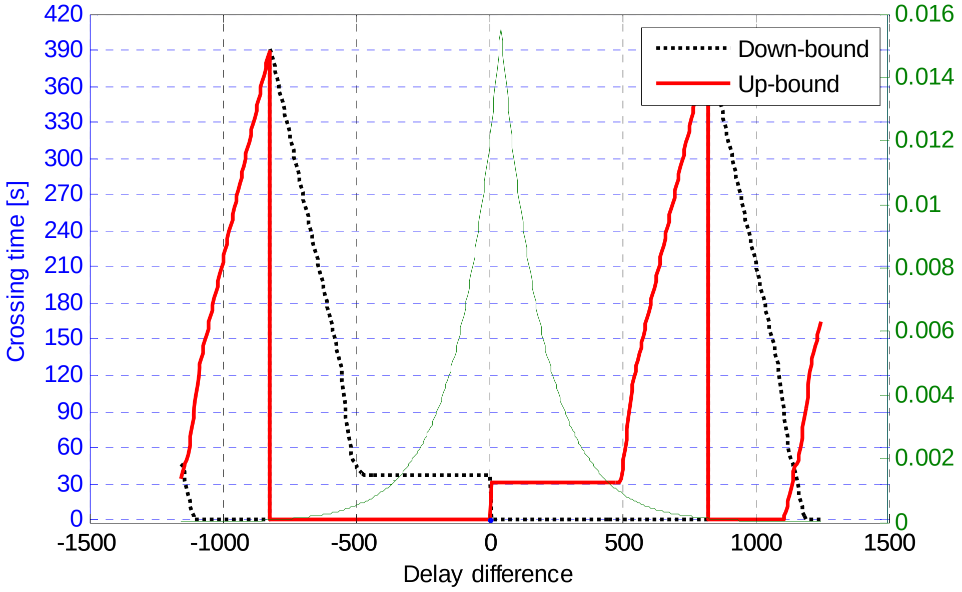
4
Max speed: 250 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 10 000 m



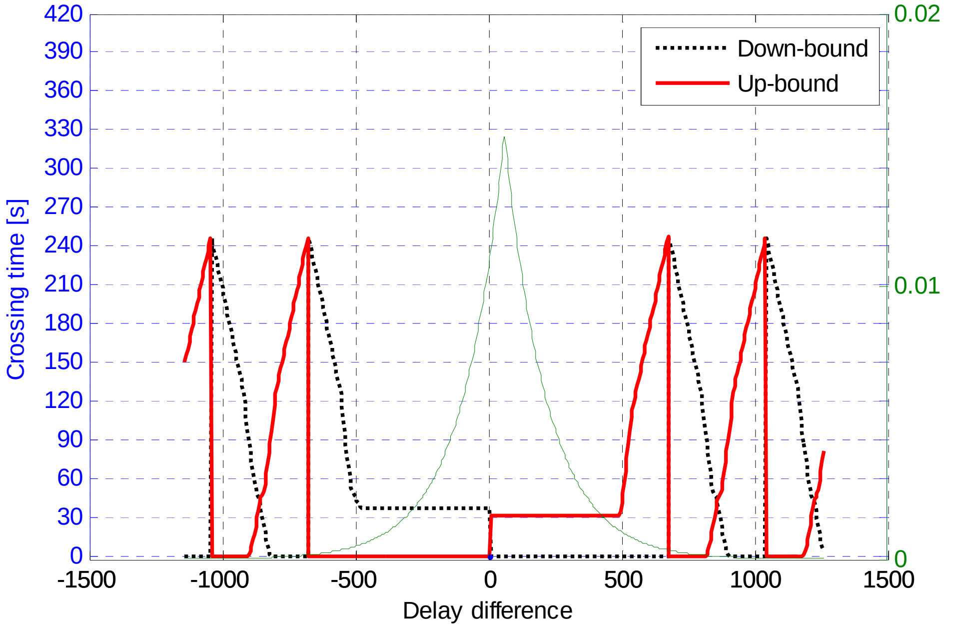

5
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 20 000 m
Crossing time [s]
420
390
360
330
300
270
240
210
180
150
120
90
60
30
0
-1500
-1000
-500
0
500
1000
1500
Delay difference
0.02
0
Down-bound
Up-bound

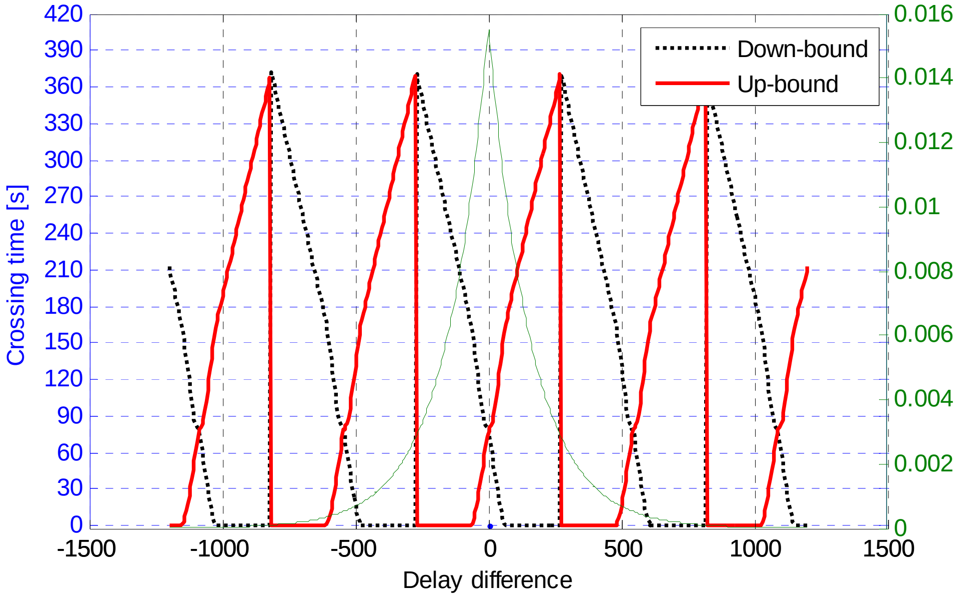

6
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 10 000 m
Crossing time [s]
420
390
360
330
300
270
240
210
180
150
120
90
60
30
0
-1500
-1000
-500
0
500
1000
1500
Delay difference
0.02
0.01
0
Down-bound
Up-bound

7
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 20 000 m

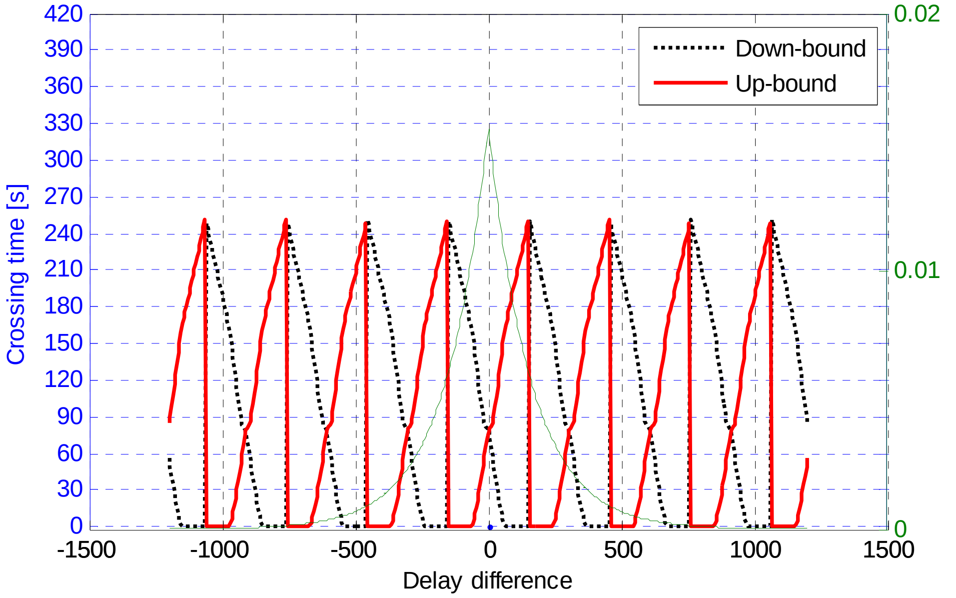


8
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 10 000 m

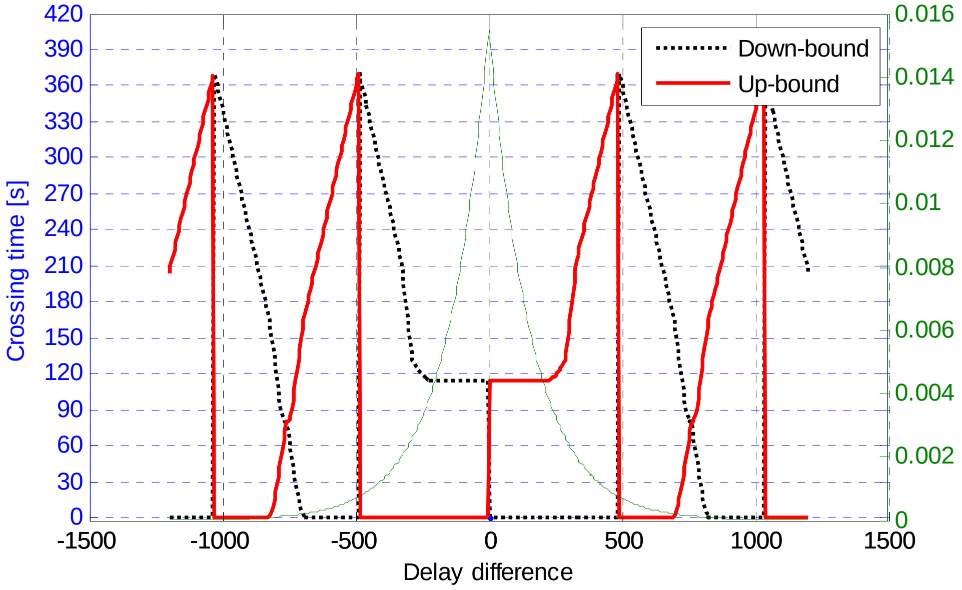




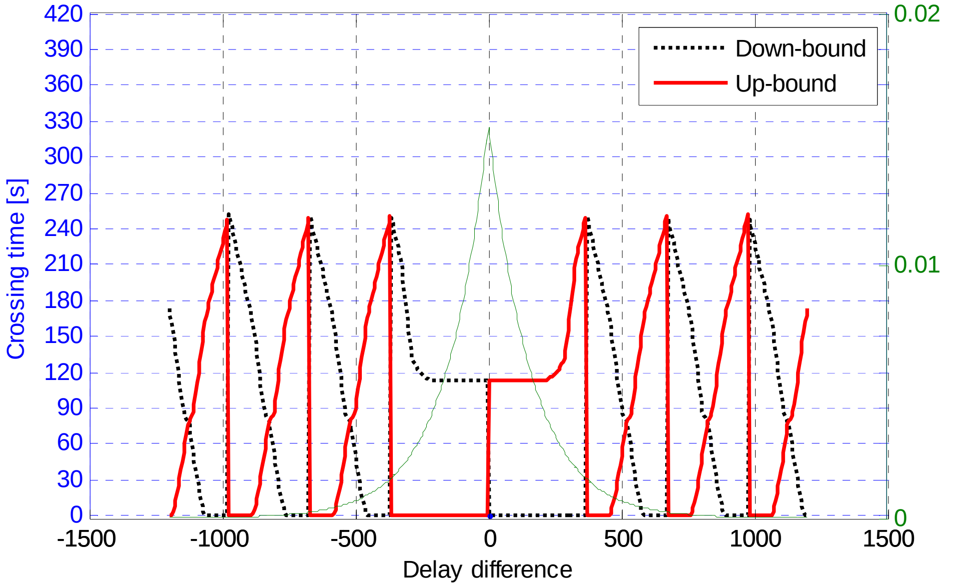
9
Max speed: 300 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 20 000 m



10
Max speed: 300 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 10 000 m

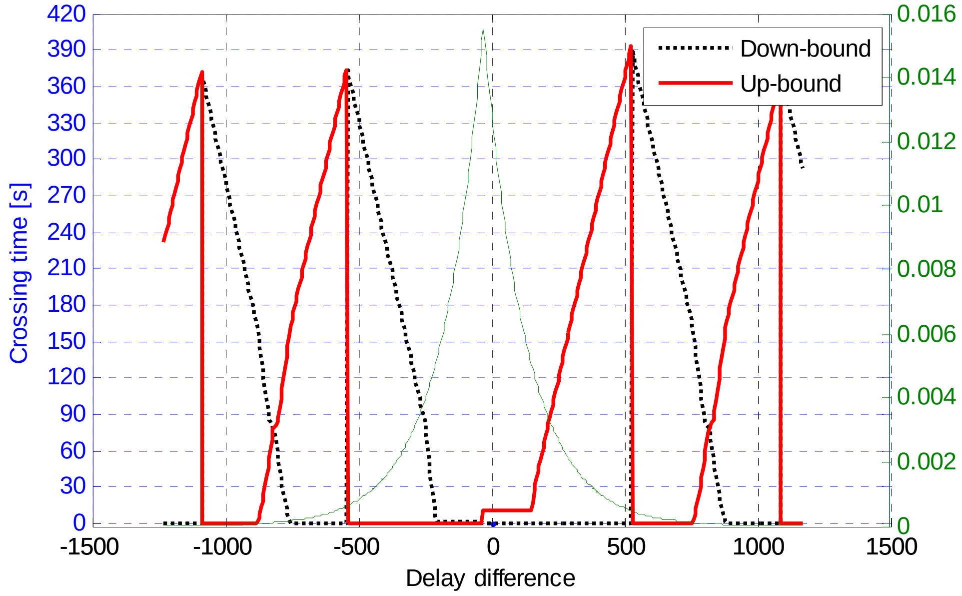


11
Max speed: 300 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 20 000 m

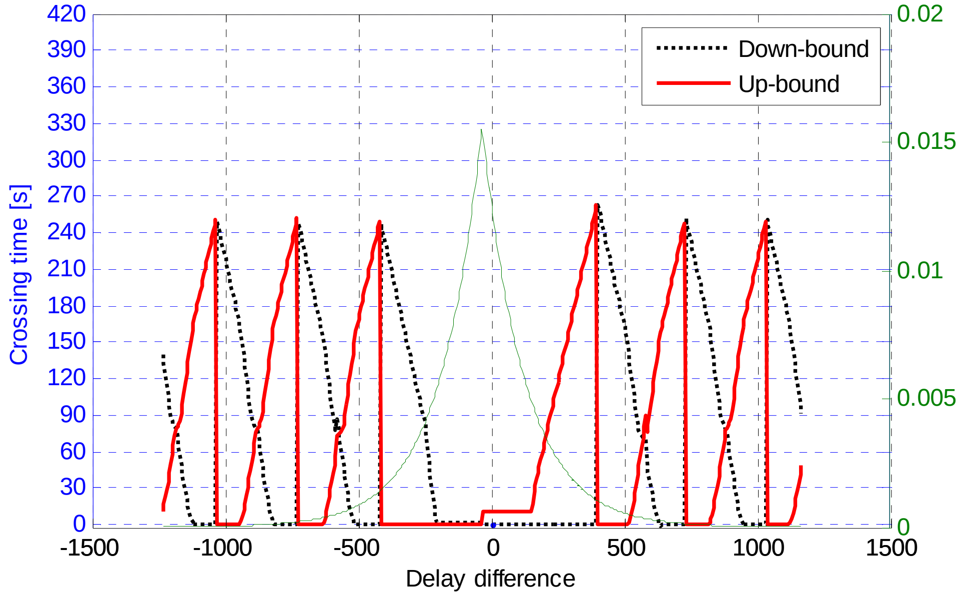


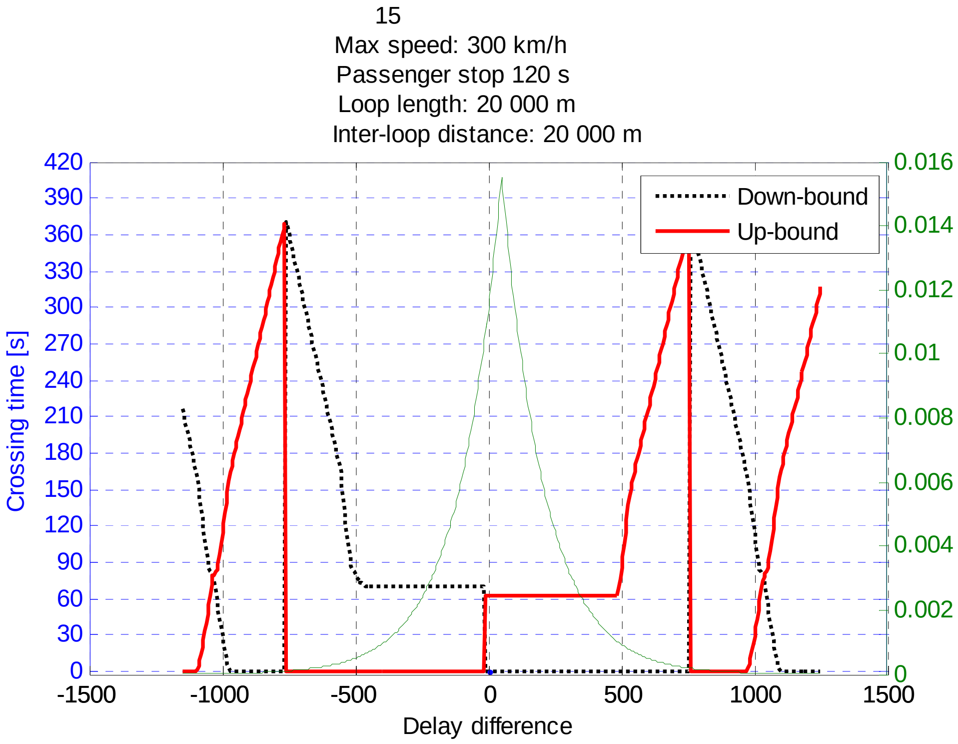
12
Max speed: 300 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 10 000 m

13
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 20 000 m

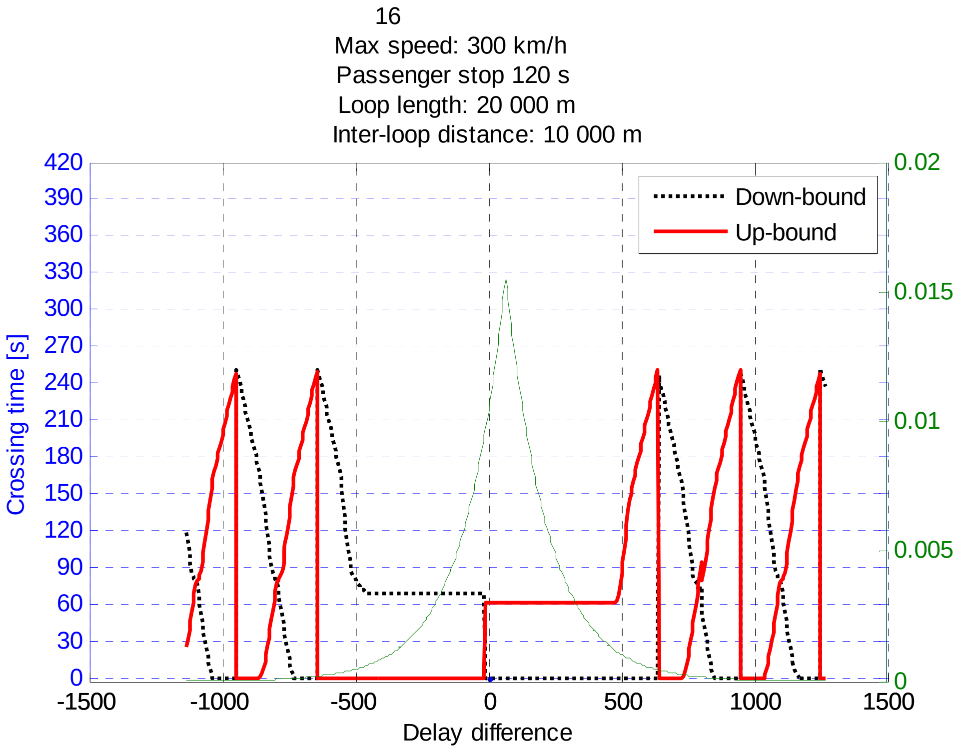


14
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 10 000 m





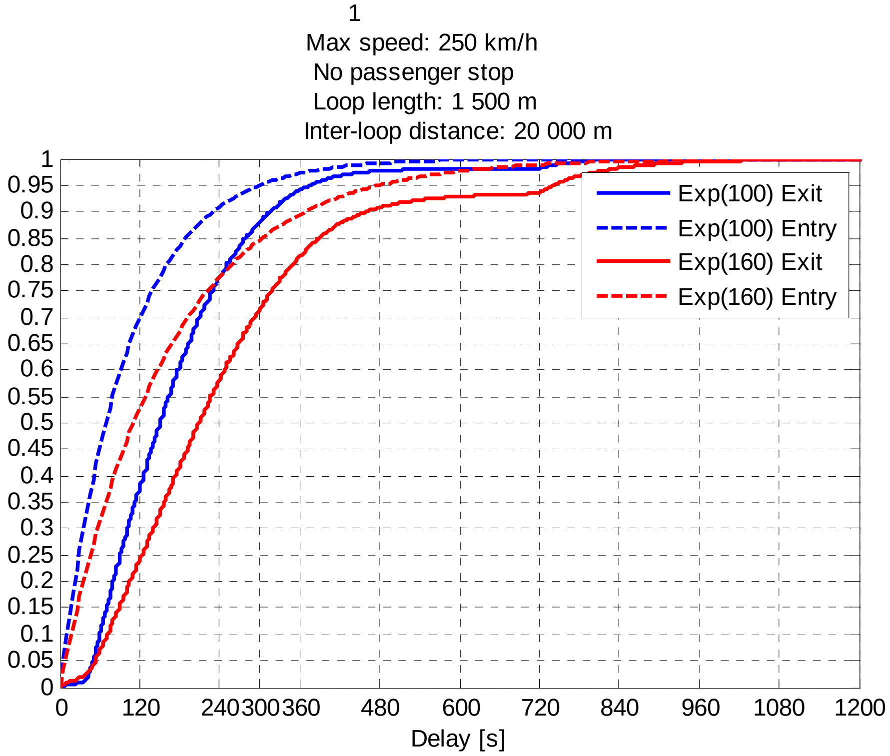
15
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 20 000 m

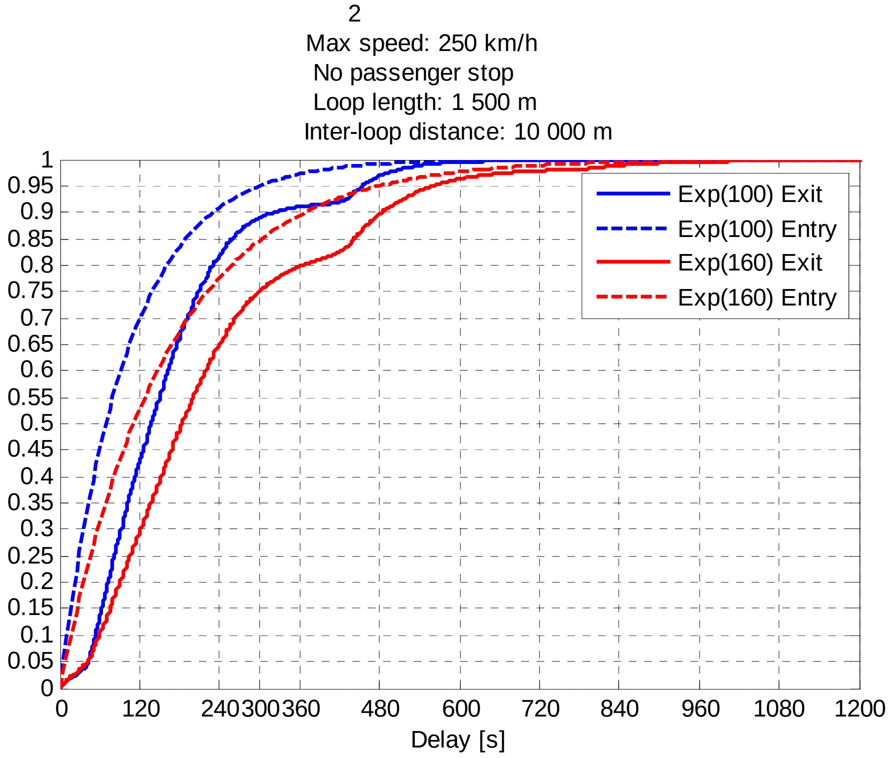


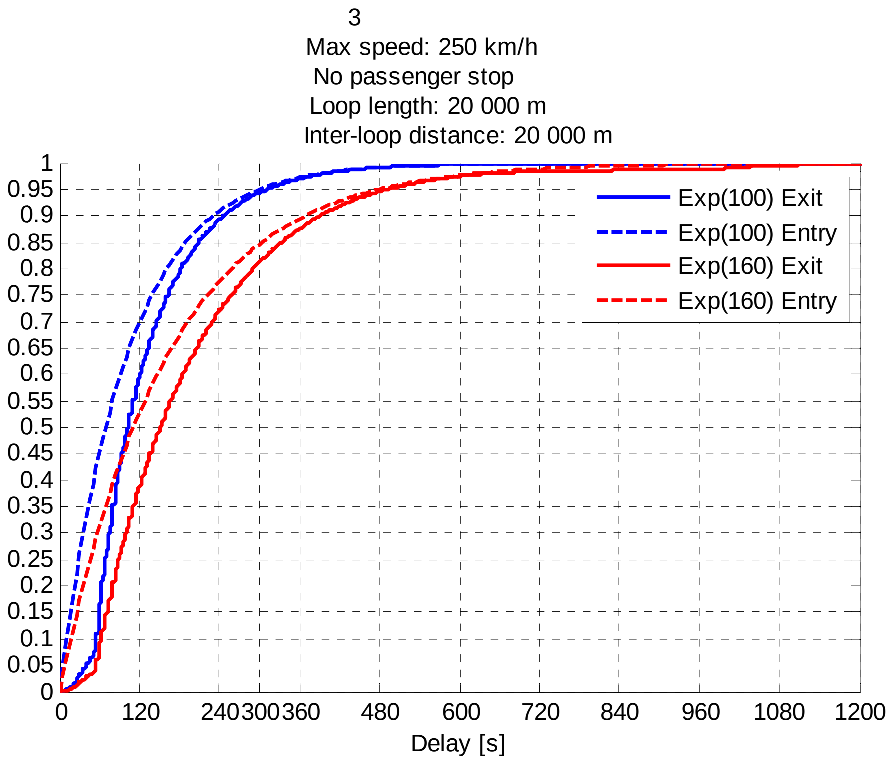
16
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 10 000 m

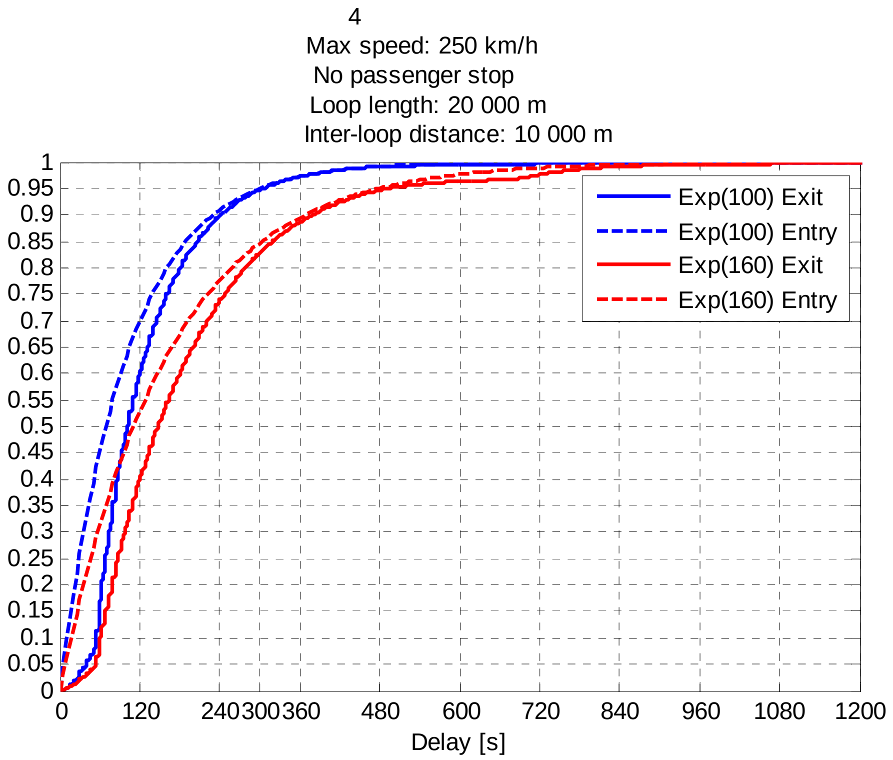
Appendix 3: Entry and exit distributions

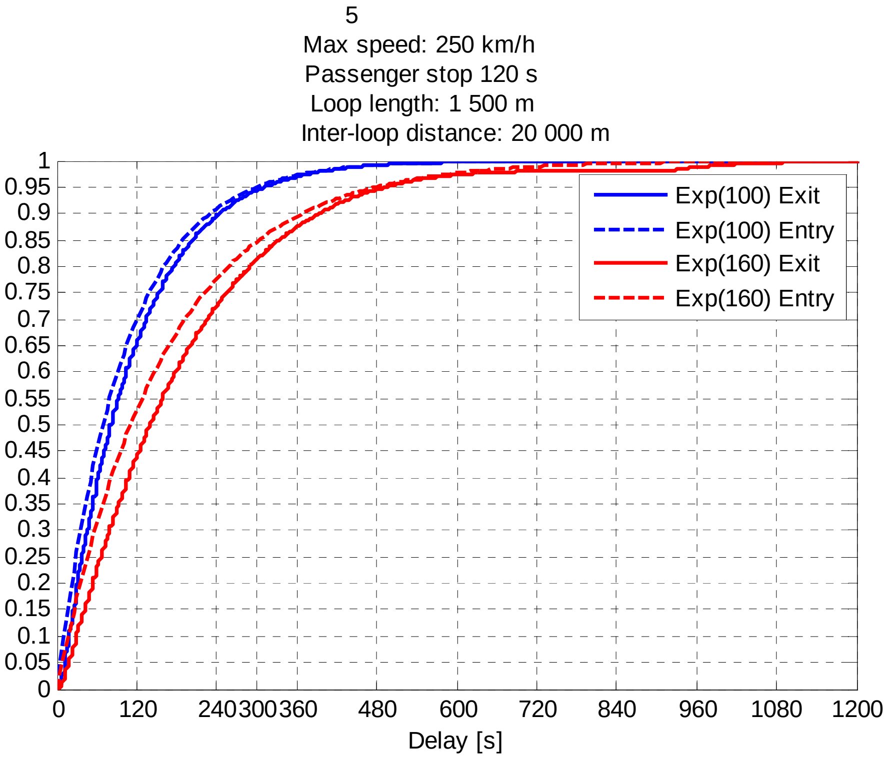
Each combination of technical factors and delay levels give rise to two exit distributions. They are shown in the following, together with the corresponding entry distributions.

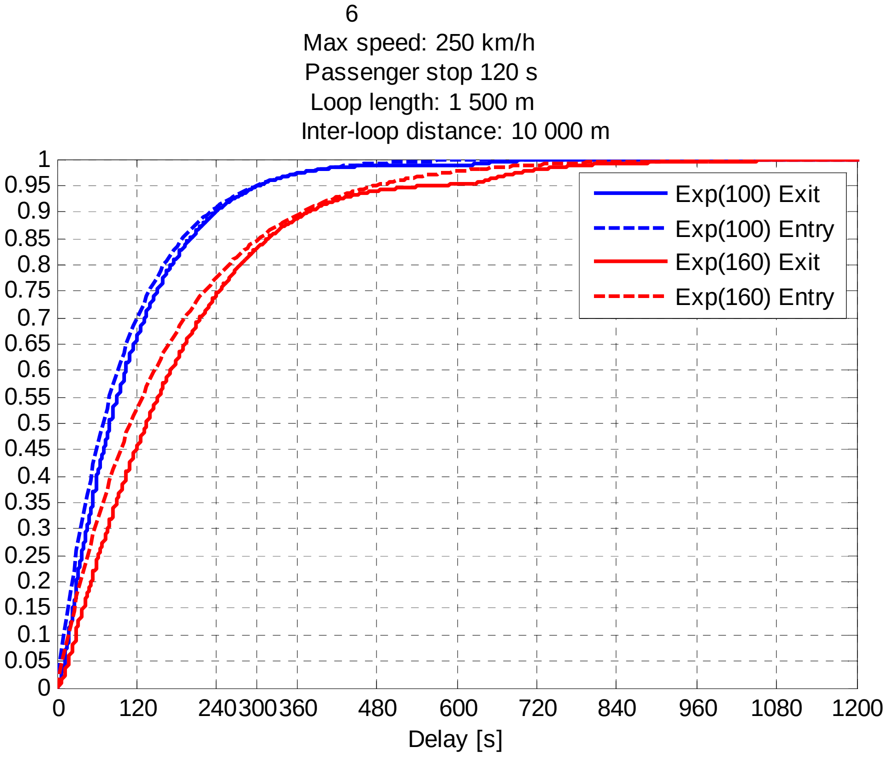

1
Max speed: 250 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 20 000 m


2
Max speed: 250 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 10 000 m

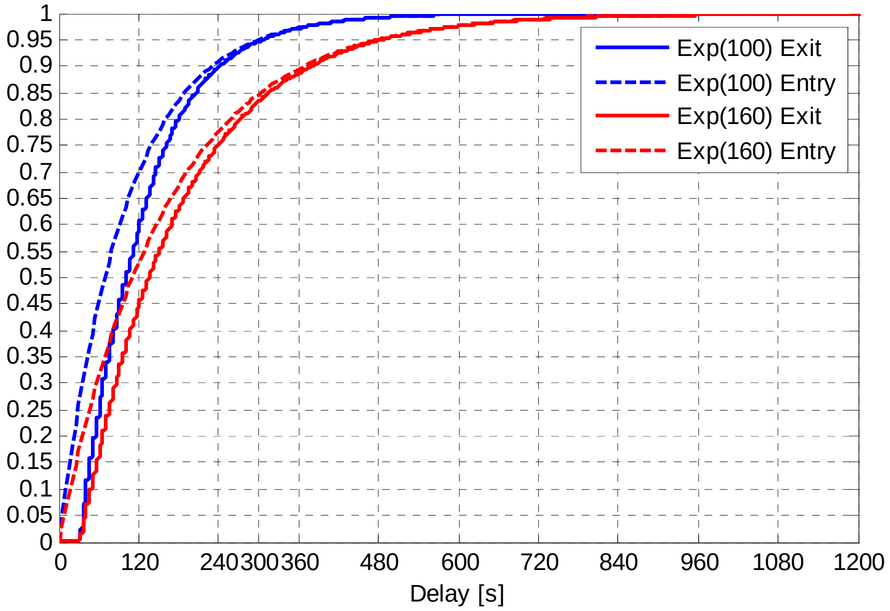

3
Max speed: 250 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 20 000 m
1
0.95
0.9
0.85
0.8
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
120
240
300
360
480
600
720
840
960
1080
1200
Delay [s]
Exp(100) Exit
Exp(100) Entry
Exp(160) Exit
Exp(160) Entry


4
Max speed: 250 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 10 000 m
1
0.95
0.9
0.85
0.8
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
120
240
300
360
480
600
720
840
960
1080
1200
Delay [s]
Exp(100) Exit
Exp(100) Entry
Exp(160) Exit
Exp(160) Entry

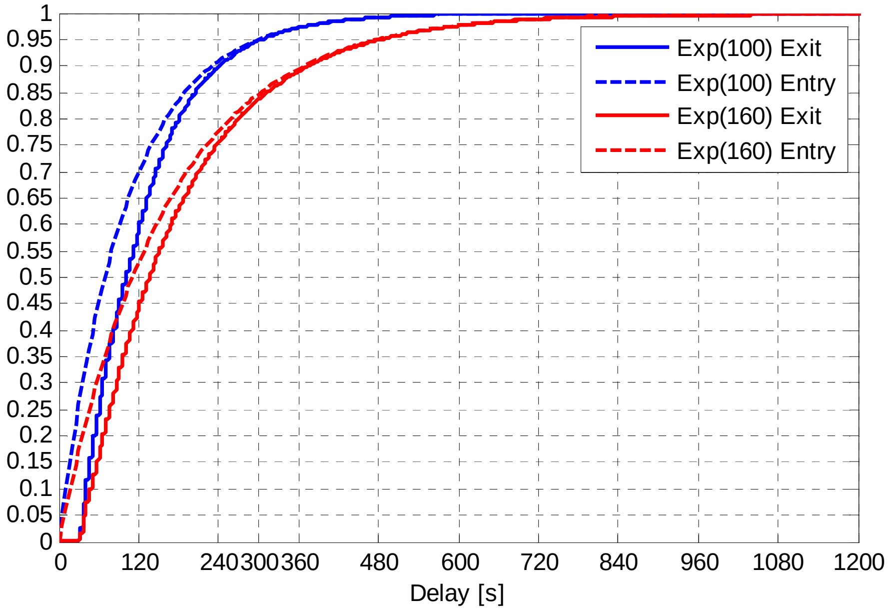

5
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 20 000 m
1
0.95
0.9
0.85
0.8
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
120
240
300
360
480
600
720
840
960
1080
1200
Delay [s]
Exp(100) Exit
Exp(100) Entry
Exp(160) Exit
Exp(160) Entry

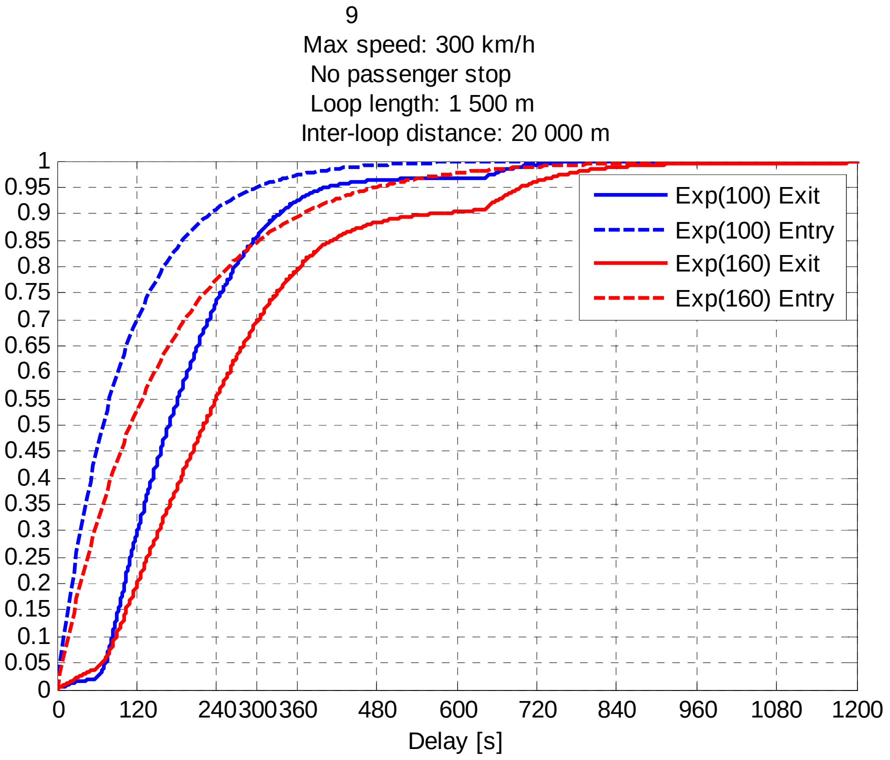

6
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 10 000 m
1
0.95
0.9
0.85
0.8
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
120
240
300
360
480
600
720
840
960
1080
1200
Delay [s]
Exp(100) Exit
Exp(100) Entry
Exp(160) Exit
Exp(160) Entry

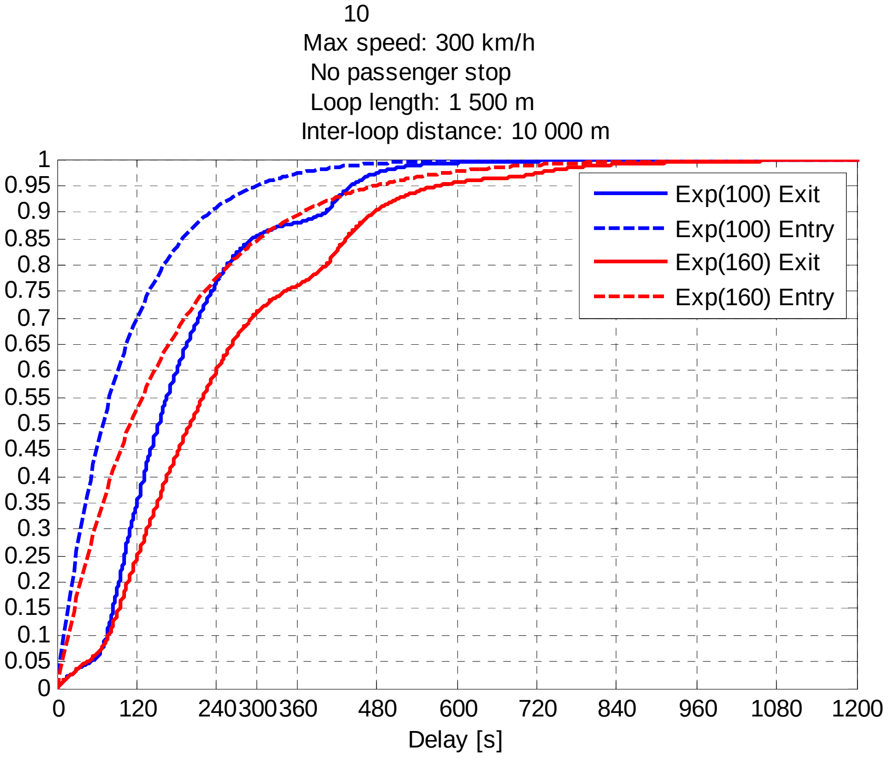
7
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 20 000 m

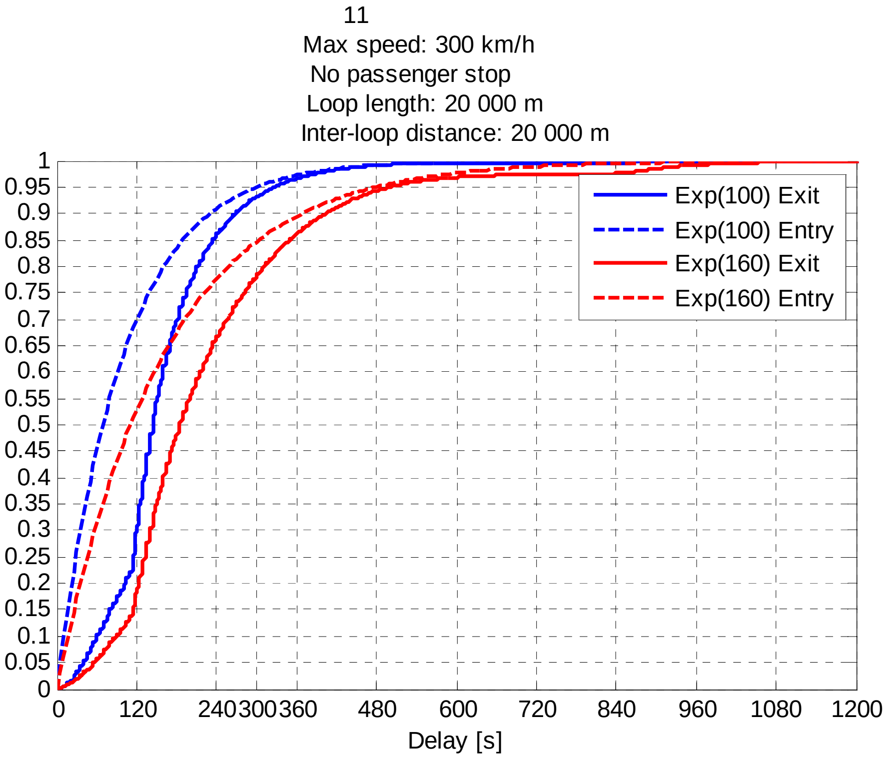


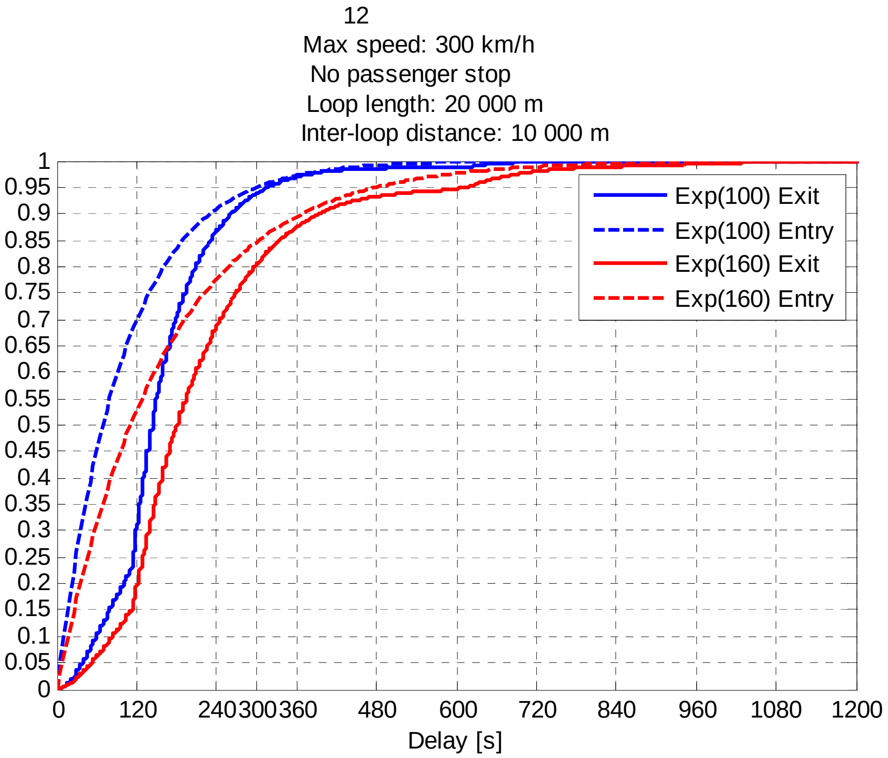
8
Max speed: 250 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 10 000 m

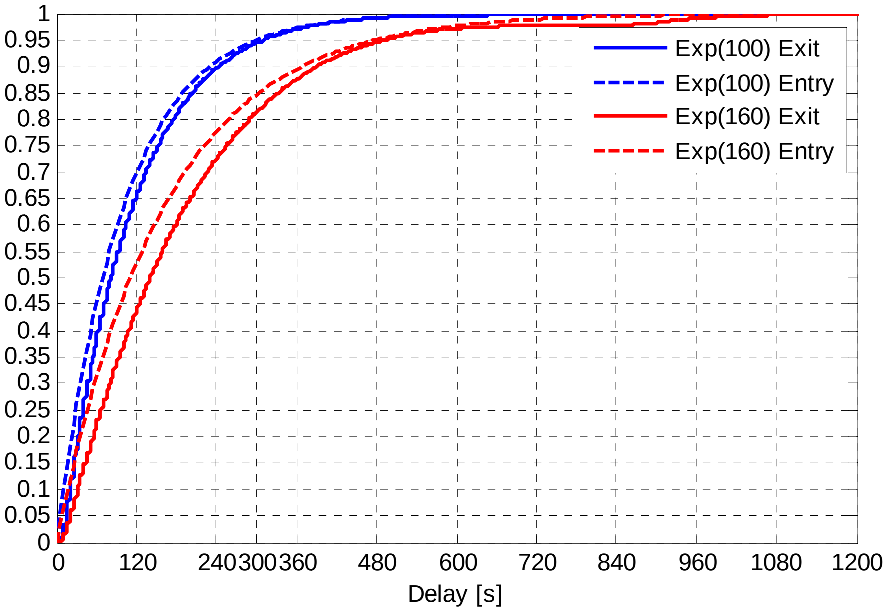



9
Max speed: 300 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 20 000 m

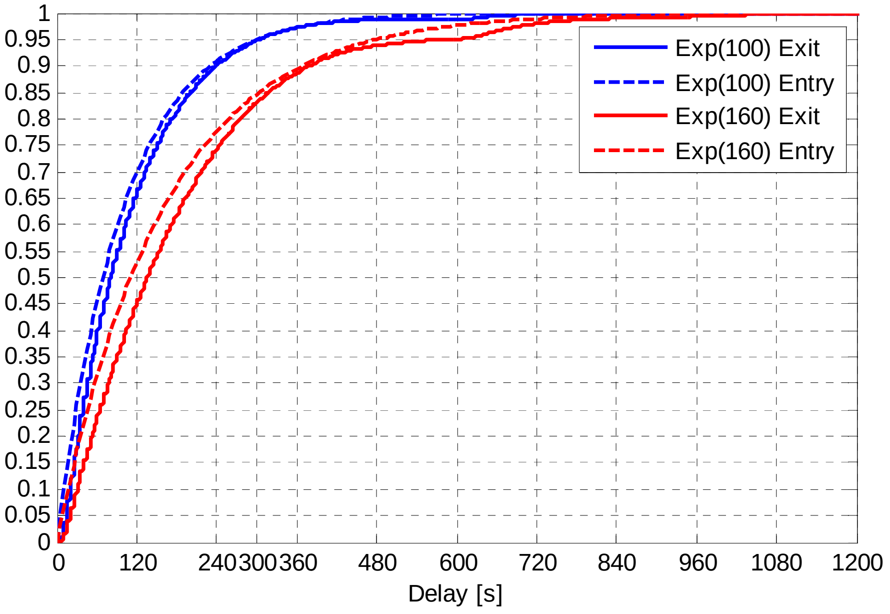

10
Max speed: 300 km/h
No passenger stop
Loop length: 1 500 m
Inter-loop distance: 10 000 m


11
Max speed: 300 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 20 000 m
1
0.95
0.9
0.85
0.8
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
120
240
300
360
480
600
720
840
960
1080
1200
Delay [s]
Exp(100) Exit
Exp(100) Entry
Exp(160) Exit
Exp(160) Entry

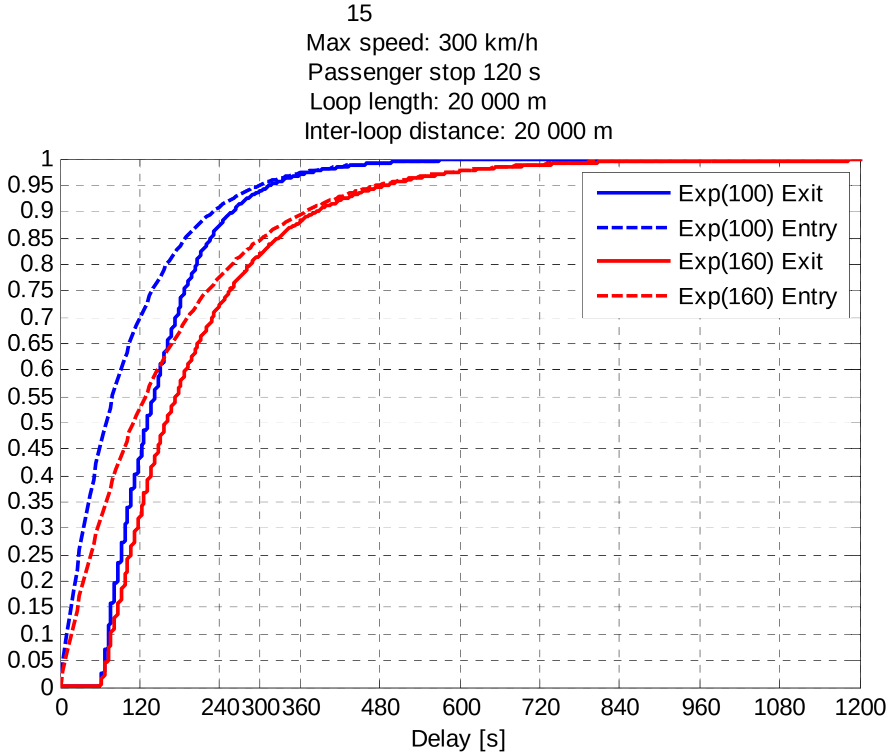

12
Max speed: 300 km/h
No passenger stop
Loop length: 20 000 m
Inter-loop distance: 10 000 m
1
0.95
0.9
0.85
0.8
0.75
0.7
0.65
0.6
0.55
0.5
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
120
240
300
360
480
600
720
840
960
1080
1200
Delay [s]
Exp(100) Exit
Exp(100) Entry
Exp(160) Exit
Exp(160) Entry



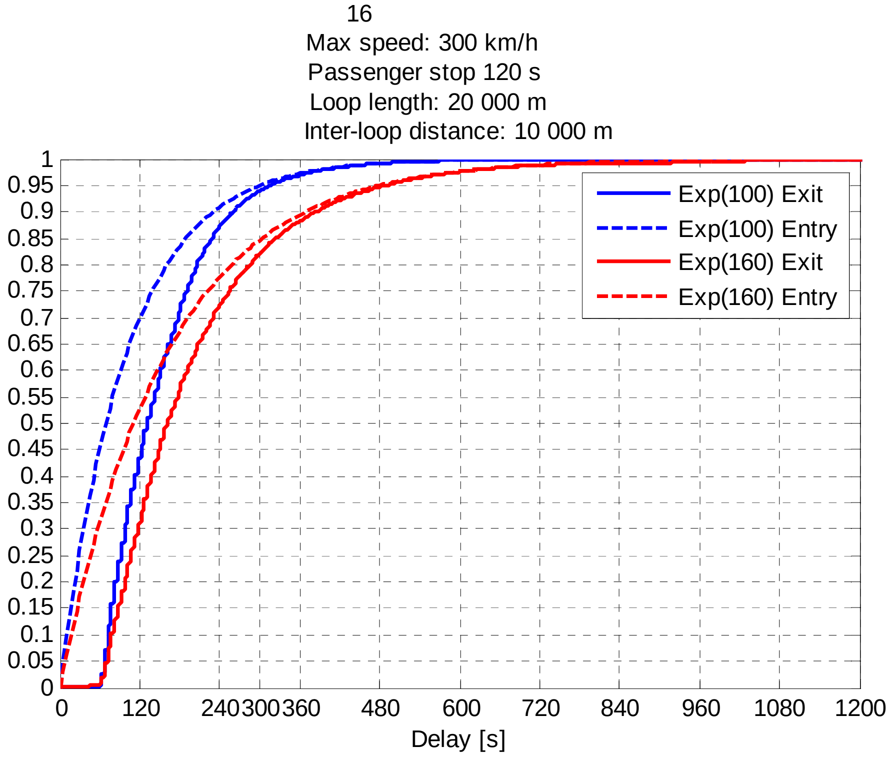
13
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 20 000 m



14
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 1 500 m
Inter-loop distance: 10 000 m



15
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 20 000 m



16
Max speed: 300 km/h
Passenger stop 120 s
Loop length: 20 000 m
Inter-loop distance: 10 000 m