Railway Operation

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Chapter 1: Characteristics of railway traffic

Railway traffic offers a number of characteristics which are very different from both road traffic and other kinds of public transport (in this case bus traffic). An important difference between a road and a railway infrastructure project lies in the consequences of the projects. Primarily, because the characteristics of road traffic differ considerably to those of railway traffic. Road traffic is (essentially) individual traffic, whereas railway traffic is exclusively public transportation. With respect to the travellers, the infrastructure therefore only appears indirectly in the lines/routes available to the traveller. Furthermore, the use of the lines/routes is paid directly to the transport company in the form of fares.

At first sight, planning of railway traffic has a number of features in common with bus traffic (lines, connections, regular frequency, regular interval timetables, etc.). However, this does not mean that the consequence calculation models used for bus traffic can be used directly. This is partly because busses are considered road traffic, partly because bus traffic has a relatively high frequency and that the traveller often has alternative routes. That is seldom the case of railway traffic, and part of the approximations that are used for calculations of bus traffic will be too rough for calculation of railway traffic. Railway systems have much more bindings in the form of safety systems, overtaking possibilities, etc.

Railway traffic (and modelling hereof) has also considerably more discreet phenomena than road and bus traffic, i.e. it is difficult to assume linear correlations. An example is the correlation between departures and available seats. For a given size of bus, more seats also mean more departures, i.e. there is a rough linear correlation between the number of departures and the number of seats. This correlation is not necessarily found in railway traffic, where the number of seats (in Denmark) can be five doubled without changing the frequency. This is explained in more details later.

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1 By bus traffic is primarily meant bus traffic in Greater Copenhagen and other big cities. The majority of the examples of cost-benefit analyses, where the impact on the bus traffic is part of the evaluation of a project, are from Greater Copenhagen (e.g. “Project Basisnet [7]”). As regards the frequency and the simple route network, railway traffic in Denmark can with respect to frequency and the simple route network best be compared with the regional bus network. The railway traffic on the Danish railway network consists of approximately 30 lines with one hour service, i.e. a route network equivalent to the regional bus traffic on the island of Funen.
Characteristics of railway traffic

1.1 Overview

Railway traffic consists of passenger trains and freight trains which on their part can be divided into different train services. There are several types of divisions, but normally passenger trains are divided into the following train services (or products):

- Long-distance trains, i.e. intercity trains, high-speed trains and international trains
- Regional trains (often fixed interval timetable all day through)
- Peak hour trains (periodic regional trains)
- Suburban lines

The division reflects the travel segment towards which the train class is targeted. Therefore the word “train product” is also used synonymously. The individual segments have different preferences with respect to frequency and travel time (and price), and it forms the background for establishing the train frequency, stopping pattern and travel time in the timetable. It is not an unequivocal definition, but the train class is a guideline for these properties. Long-distance trains are non-stop trains and they only stop at major stations in order to obtain a high travel speed. The property is related to the services and is therefore primarily an information to the customers, e.g. about compulsory reservation.

According to this definition it is necessary to distinguish between train class and rolling stock type. There is not always identity between certain train services (departures) and rolling stock. For instance, both diesel and electric rolling stock can be used for intercity departures. The two rolling stock types have different driving behaviours, train capacity, etc., so the operational costs will vary. When calculating the consequences, this issue must be considered too. In a system with regular interval timetables, the travel time is planned according to the rolling stock type with the worst properties.

A particularly important aspect is that the number of “vehicles” per departure may vary from one departure to another. This gives a fundamentally different correlation between seats available and frequency than what is the case of bus traffic (where you can e.g. choose an articulated bus). The impact calculation of a railway project must therefore be done in a quite different way.

The composition of the railway traffic, i.e. the mix of different train services, will vary between the lines within the railway network, and thereby the appearance of the timetable. It also means that the distribution between freight and passenger transport, including the distribution between long-distance trips, commuter traffic and leisure trips, can vary considerably between the lines.
Thus, the impact calculation will often differ from one project to another, even though the infrastructure extension carried out is the same (e.g. extension to double track). Project evaluations of railway projects are therefore relatively project specific.

In the following section, the following aspects are discussed:
- The capacity conditions affect travel time and frequency
- The importance of calculating the rolling stock
- The traffic is planned. An independent planning assignment
- The future timetable is partly unknown
- The utilization problem (i.e. are the effects realized?)

1.2 Capacity conditions

The capacity conditions in railway traffic are fundamentally characterised by the very limited overtaking possibilities of the individual trains. This property implies that the travel time for one train may influence the travel time of other trains, and that the travel times will therefore depend on the actual timetable. Railway traffic can be compared with a long bus lane where transit buses must only overtake at selected stops. This lack of continuous overtaking possibilities gives rise to many dependencies between the individual train departures. These dependencies are partly seen during the planning phase, where they have a great influence on the design of the timetable, i.e. the establishment of the departure and travel times of the trains, and partly during the operation process as delays spread to other trains.

Despite the similarities between the planning of bus and railway traffic, the capacity conditions between road and railway differ so much that the modelling of travel time will be very different. The capacity conditions of railway traffic also mean that the way in which a capacity problem is recognized, differs considerably from road traffic. The classical point of view is: “How can there be 10 minutes between each train and still be capacity problems?”

The explanation is that capacity problems are not only recognized at a line segment or a cross section of the infrastructure\(^2\). In figure 1.1 is seen a graphic timetable (a distance-time curve) for a section of a railway line. The necessary time interval between the trains at this place – the headway – is 3 minutes, so it appears that there is room for other trains. It can also be seen that these trains can have several different timetables without it giving rise to capacity problems – the

\(^2\) However, for urban lines with homogeneous traffic it will however be possible
Characteristics of railway traffic

trains can be placed in the graphic timetable in several ways without conflicting with the other trains.

The same trains form part of figure 1.2 that however shows the entire line between some major junctions, H-city and K-borg. When we look at the trains in these cross sections, it is seen that they are only separated by the minimum headway time. In other words, the trains are placed as close to each other as possible. In this imaginative example, it is not possible to introduce further trains on the line. If more trains are to be introduced on the line, it requires an overtaking along the line together with free platform capacity at the junction stations at the relevant times. Alternatively, the average speed of the trains (in the timetable) must be homogeneous. Whether this is possible/desirable entirely depends on the actual local conditions.
The example shows that the capacity problems are due to conditions away from the cross section which was originally studied in figure 1.1. Generally, a cross sectional consideration is not sufficient to recognize the capacity problems. As a main rule, it is necessary to observe lines between major junction stations, where the railway traffic changes, to get a reasonable picture of the capacity conditions. In case of dense traffic on a line, it can also be necessary to observe several adjacent lines when studying the capacity conditions. The traffic on the remaining network can also have a considerable influence on the traffic on a line or a station.

Apart from the fact that the capacity conditions influence the travel time (e.g. by introducing an overtaking), figure 1.2 also illustrates that the capacity conditions can influence the frequency in case of dense traffic. If several adjacent lines and stations are close to capacity level, it is not necessarily easy to introduce several trains in the timetable. The possibilities will depend on the actual conditions. This means that the present availability of train departures – and thereby the frequency on a line – is not only limited by the demand, but maybe also just as much by the capacity conditions of the infrastructure

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3 An example hereof is the line Copenhagen – Holbæk, where DSB has introduced double-deck rolling stock. Alternatively, the number of departures could be increased. However, this possibility requires a sufficient capacity of the infrastructure.
Characteristics of railway traffic

The example also shows that there are several methods to solve a given capacity problem. In the short term it may be possible to change the timetable. In the somewhat longer term it is possible to purchase rolling stock or the infrastructure may be extended, or combinations hereof can be used. The same goes for the methods to reduce the travel time. In this case there are several possibilities which to a certain extent can be substituted. In Switzerland for instance, it has for certain lines been decided to extend the infrastructure and for other lines to insert tilting trains to obtain travel time reductions as part of the Bahn2000 project [4].

The example in figure 1.2 is a typical example of the conditions on double track lines, where the traffic is a mixture of “fast” and “slow” trains, normally synonymously with “transit” and “stopping” trains, respectively.

The average speed of a train (in the timetable) is determined both by its stopping patterns and by the maximum speed of the rolling stocks as well as the acceleration properties (and of course by the permitted speeds of the infrastructures). The speed differences for trains are a general phenomenon, but at the same time the distance between junction stations can also vary (i.e. the stations, where it is planned that the fast trains overtake the slow ones). The range in the distances between junction stations can for instance be observed on the railway from Elsinore to Copenhagen (Kystbanen) between stopping and non-stop trains and on the line Copenhagen – Århus between high-speed trains and long-distance multiple unit train.

The most common way to increase the capacity (i.e. the possibility to introduce more Re-trains) is to extend the travel time for the fastest trains. By harmonising the speed, the capacity is increased at the expense of the travel time of the fast/non-stop trains.

The capacity and travel time conditions are different on single track lines. The trains going in opposite directions are mutually dependent and can only pass (cross) each other at selected stations. As the crossing stations are not necessarily situated where the trains naturally meet, the crossing can take place at the nearest station, which can give rise to waiting time. How much will depend on the actual timetable.

For a more profound description of the capacity conditions for single and double track lines as well as stations, reference is made to later chapters. Thus, as regards consequence calculation both the travel time and partly the frequency depend on the capacity conditions. The most common method to include capacity conditions is to work out a graphic timetable (distance-time curve) that respects headway, crossing possibilities, etc.

In the literature, capacity is often defined by traffic intensity, a number of trains per time unit at a given point of the infrastructure. In general,
this number is attached with further conditions, e.g. a special mix of different train services. But in the light of the relatively big influence of the timetable on the travel time, an alternative capacity concept can be defined:

### Capacity can be defined as:

The capability of the infrastructures to handle one or several timetables.

It is difficult to handle this definition quantitatively without further requirements to these timetables, but qualitatively the definition reflects that when the infrastructure project has been finished, the traffic will be planned according to a timetable. This timetable has properties with regard to travel times, wanted departure times and frequency (and regularity) that depend on the capacity of the infrastructure.

As a last comment on railway capacity conditions, it should be noticed that queues are not directly visible and are therefore more difficult to measure than in case of road traffic. It is therefore more complicated to use queue phenomena in the existing traffic as a planning tool. As railway traffic is planned, part of the queues will be eliminated beforehand as the rejected trains are not included in the timetable. Elaboration of a timetable is therefore also an evaluation of the infrastructure capacity (train capacity is another issue).

Since railway traffic is timetabled, the queue phenomena will appear in two ways:

- Delays (i.e. a non-planned deviation from the timetable)
- Travel time extensions (i.e. extra time included in the timetable)

As regards queuing theory both elements give rise to waiting time, but it *is valued* in different ways by the travellers. Travel time extensions occur in the planning phase, delays in the operation phase. These aspects have particularly been treated in the German capacity literature (cf. [5] for an overview).

Traditionally, changed regularity is not calculated in connection with the project evaluation (an exception can be found in [6]). This is due to several reasons: Partly because the punctuality cannot be calculated analytically, as this requires simulation of the very complicated interactions between traffic and infrastructure, and partly because travel time and punctuality are substitutable to a certain degree, and therefore it is an advantage to keep one of these constant when making the evaluation. When planning next year’s timetable it is
however an extremely relevant issue, but it is not within the scope of this chapter to deal more detailed with the subject.

1.3 Rolling stock and train capacity

Railway operation requires much capital, and procurement of rolling stock is often an important cost of investment. Apart from the other operational costs, it is therefore important to calculate the changed need for rolling stock. For instance, one IC3 multiple unit costs 40-45 mio. DKK (the train unit has approx. 130 seats, equivalent to the capacity of 2 – 3 busses). The working life is often 20 – 30 years as opposed to 10-12 years for busses. Due to the ongoing technological development during these 20 – 30 years, new rolling stock both gives better comfort and better driving behaviour than the stock it replaces.

Generally, the replacement of rolling stock offers a possibility to reduce travel times and to obtain timetable improvements. The investments in procurement of rolling stock is often just as big as the investments in infrastructure projects, and the projects have the same realization time (2 – 4 years). But some times replacement of rolling stock is an alternative to short-term improvements of the infrastructure. That is especially the case in Denmark, where passenger trains drawn by locomotives are replaced by motor-driven multiple units that offer a considerably better driving behaviour.

Historically, it has been particularly important to calculate investments in rolling stock separately, as it was financed via the state budget in line with the infrastructure. In the new organisation of the Danish railway sector (with the State as traffic buyer), rolling stock will in future be financed by the operator (DSB and Railion among others) by means of borrowing, rent/leasing, and this item will be a substantial part of the train operating charges. It complicates the prerequisites for a socio-economic analysis.

Operation planning operates with 2 concepts with respect to rolling stock needs:

- trains
- train unit/wagons

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4 This is one of the fundamental ideas in DSB’s plan “Good trains for everybody” [3]. However, the plan is primarily a reinvestment that has been moved forward, as the rolling stock used in the regional traffic on Zealand is 25 – 30 years old.

5 The travel time of trains drawn by locomotives depends on the number of wagons attached to the locomotive, whereas this effect can be disregarded in the case of train units. In time tables with trains drawn by locomotives, travel times are determined based on the largest number of wagons occurring during the week/day, i.e. relatively slow travel times. The introduction of train units therefore results in substantial time gains, as the travel time is the same, no matter how many train units the train consists of.
A “train” is defined as the rolling stock used in a departure. There is one train in each departure. The number of necessary trains depends among other things on the timetable, and the necessary number of trains can therefore be calculated directly based on this. A train can either consist of one or more-train units or a number of passenger wagons drawn by a locomotive (passenger trains drawn by locomotives are losing importance in Denmark).

A “train unit” is a motor-driven unit that can be attached to other train units. In Denmark, a train can thus consist of 1, 2, 3, 4 or 5 train units depending on the rolling stock type. Provided there is only one train unit per departure, the number of trains and train units is identical. The number of train units needed therefore partly depends on the number of trains required by the timetable, but also on the requested number of train units per departure. A certain knowledge about how the demand is distributed on departures (hours) is therefore necessary.

The Copenhagen suburban service represents a quite simple example of calculation of the number of train units. In the morning and afternoon peak-traffic hours, a typical departure consists of 2 train units, and during the rest of the day and the weekends it normally consists of 1 train unit. On the remaining railway network, there are major variations in the number of train units per departure during the day and the week.

In the below table is shown the variation in number of seats per departure for different rolling stock types.

<table>
<thead>
<tr>
<th>Rolling stock type</th>
<th>Capacity per train unit</th>
<th>Train units per departure (train length)</th>
<th>Possible seats available per departure</th>
</tr>
</thead>
<tbody>
<tr>
<td>MF (IC3 long-distance trains)</td>
<td>138</td>
<td>1 – 5</td>
<td>138 – 690</td>
</tr>
<tr>
<td>ER (Odense, etc)</td>
<td>222</td>
<td>1 – 4 (5)</td>
<td>222 – 888</td>
</tr>
<tr>
<td>MR (Jutland/Funen)</td>
<td>132</td>
<td>1 – 5</td>
<td>132 – 660</td>
</tr>
<tr>
<td>ET (Øresund trains)</td>
<td>237</td>
<td>1 – 4 (5)</td>
<td>237 – 948</td>
</tr>
<tr>
<td>SA (S-trains)</td>
<td>336</td>
<td>1 – 2</td>
<td>336 – 672</td>
</tr>
</tbody>
</table>

Table 1.1 Available seats for different rolling stock types [2]

6 Until year 2005.
Characteristics of railway traffic

Thus, when calculating the consequences an extra dimension must be taken into consideration as compared to bus traffic, i.e. that the number of seats per departure varies considerably more than in case of bus traffic. The introduction of extra train units on a departure is the most frequent way to handle the timely variation in the demand in case of railway traffic. Alternatively, the frequency is increased if possible.

In case of bus traffic, it is possible to introduce a bigger bus type, but the introduction of more busses also means that the frequency is increased. It is therefore more natural to establish an approximation to a linear correlation between the number of travellers and the frequency of the bus traffic.

Local conditions determine whether train units are introduced in existing departures or used to increase the frequency by introducing an extra departure. The limiting factor is often the infrastructure capacity. In figure 1.3 are shown two ways to increase the available seats: the introduction of extra train units to obtain the maximum train length and the introduction of several train units by increasing the frequency, respectively. 800 travellers per hour can therefore be equivalent to 1, 2, 3 or 4 departures, respectively.
It is characteristic that the need changes discreetly, and due to the price of the rolling stock it is a significant effect, if a project makes it possible to reduce the use of rolling stock by just one train unit.\footnote{One of the essential gains from high-speed trains is precisely the better productivity of the rolling stock.}

The frequency is often determined beforehand based on service considerations or on a rough estimate of the demand. Afterwards, the train capacity is dimensioned, i.e. the necessary number of train units per departure. Thereby the total need of rolling stock can be calculated. When calculating the consequences of railway traffic, 2 types of transport work occur:

- train-km
- train unit-km
Characteristics of railway traffic

The first is related to the number of departures and is equivalent to the transportation work with trains (1 train unit in each departure). The amount of transportation measured in train unit-km is related to the total number of km driven with all the train units, and thereby to the dimension of the seat capacity. If trains drawn by locomotives are used, train unit-km are replaced by wagon-km.

The two types of driving work are used to calculate the different types of effects.

1.4 The level of service is part of the alternative

Traffic forecasts play a central role when evaluating infrastructure projects. Traffic forecasts describe how the project influences future travel flows, and based on these data the effects for the travellers and the surroundings are calculated. In case of big projects, traffic models can be used as tools to work out forecasts.

Traffic forecasts can partly be used to establish future travel flows of persons and partly to establish the number of vehicles and their distribution on the routes within the network, i.e. the traffic flow of the infrastructure. The number of vehicles and their driving work is an essential input for a number of impact models (for road as well as railway traffic). When evaluating road projects, an average occupation factor per car can be used, and it is therefore relatively simple to convert from number of trips to number of cars. In the literature, the expression traffic forecasts is therefore often used synonymously for both future number of trips and vehicles in the network, respectively.

For railway projects it is necessary to prognosticize the future timetable, i.e. lines, frequencies, departure times etc. It makes no sense to prognosticize this by means of the usual techniques, as the future traffic supply is a result of planning and choices made by operators, infrastructure managers and traffic buyers. As railway traffic is a public means of transport, traffic and supply is planned on beforehand. Planning of the traffic on the existing infrastructure is already a comprehensive planning assignment.

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8 By means of the number of vehicles etc., the influence of the capacity conditions on the average travel time can be modelled and the demand for transport may consequently be influenced. Precisely this condition cannot be modelled in the same way for railway traffic.

9 Traditionally, the planning process has taken about 1 year from the first project, co-ordination with all the regional transport authorities to the final production plans for rolling stock, locomotive and train crew and integration in sales systems.
For consequence calculation processes regarding a new railway project this means:

- that the operation pattern has to be chosen in order to carry out the project evaluation
- that the operation pattern is representative of the infrastructure alternative.

It is specific for railway evaluations that the same infrastructure alternative can be represented by several different operation pattern proposals. There is no clear correlation between infrastructure and frequency, and for the same plan of operation there can be worked out several different candidate timetables. It means that the conclusions of a project evaluation is to a certain extent based on the chosen operation pattern prerequisites. It is obvious that the results depend on whether there is planned 30 minutes service instead of one hour service and thereby the profitability of the project in the socio-economic analysis.

In Sweden, where socio-economic evaluations of projects have been used for the last 10 years, this problem has been studied by “Riksrevisionsverket” (the Swedish National Audit Office) that has examined a number of “Banverket’s” analyses (the Swedish infrastructure manager) [8]. It is maintained in the report that forecasts are more important for railway projects than for road projects in an appraisal context. The argument is that typical road projects result in such small time benefits that the influence on the total volume is marginal, whereas we in case of railway projects talk about newly generated trips or transfer from other means of transportation (the railway projects examined all contained considerable travel time reductions). It is also underlined that it is a methodical problem that Banverket has to be able to handle the financial decisions regarding the operation pattern made by the train operator or the traffic buyer10.

When evaluating a railway project it is in reality the combination of infrastructure and operation patterns that is being evaluated, more than the infrastructure itself. If you wish to carry out a sensibility evaluation of a project, the operation pattern is thus one of the conditions that can be tested for sensibility. The expression operation patterns is used as the overall concept of the future traffic supply,

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10 The national traffic in Sweden takes place on commercial conditions, whereas it in Denmark takes place by means of a contract with the State. The Ministry of Traffic plays the part as traffic buyer in Denmark
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described by a candidate timetable\textsuperscript{11} or maybe only by means of a plan of operation\textsuperscript{12} (i.e. only frequency and lines).

As earlier mentioned changes in the operation patterns can sometimes replace infrastructure extensions. The travel time can for instance be reduced by eliminating stops for selected trains, and thereby reducing the frequency at some stations. Whether fewer stops are an alternative, depends on the project.

A substantial part of the consequence calculation is therefore to set out the operation pattern prerequisites that are expected to be present when the project is placed into service, so that the consequences can be calculated. In principle, this traffic planning assignment could be solved by going through the same steps in the planning process when planning the annual timetable, and to use the same tools. For consequence calculations, it is normally enough to analyse an average workday. It is often sufficient to analyse the traffic for one hour during the peak-traffic hours or a so-called normal hour, i.e. an hour outside the peak-traffic hours (nonetheless, the material need is determined by the need during the peak-traffic hours).

The planning of railway traffic is a hierarchical process which, in schematic form, may appear as in figure 1.4.

1. Preparation of plan of operation Line, frequency, stopping pattern
2. Elaboration of candidate timetable Timetable times, capacity control
3. Elaboration of full timetable Normal week, annual timetable
4. Calculation of rolling stock need Rolling stock roster (driving work)
5. Crew scheduling Operational costs

Figure 1.4 Planning process for railway traffic (based on [1])

The traffic planning for consequence calculation primarily lies in steps 1 and 2, and for this purpose is used simplified calculation models for

\textsuperscript{11} The word “candidate timetable” is here defined as a timetable for a selected time of the day, typically an hour, as the timetable is typically the same each hour, when regular interval timetables are used as a principle. The candidate timetable indicates that it has not necessarily been made out with the same level of detailing as the present timetable.

\textsuperscript{12} The word “plan of operation” is here used only about the line structure, the stopping pattern and frequency. The departure times are not added until the plan of operation is converted into a candidate timetable. In everyday speech, there is normally no strict distinction between the two words.
steps 4-5. When preparing next year’s timetable, emphasis is placed on steps 3-5. The marginal conditions for elaboration of next year’s timetable and timetables for project evaluation are different. But the planning order is the same in both cases. ‘Scheduling’ often uses last year’s timetable as a starting point, and the planning therefore normally consists in modifying existing plans. The infrastructure and the amount of rolling stock available is known beforehand, whereas you in case of a project evaluation is about to change the infrastructure and/or new rolling stock.

Thus, part of the uncertainty when evaluating an infrastructure alternative is due to the planner’s assumptions about on the one hand the future frequency and on the other hand the future timetable. Often, a project alternative is compared with a basic situation, consisting of the day’s traffic, where the timetable is well-known. The traffic in basis is often described more detailed than the traffic in the project alternative.

1.4.1 Local effects or network effects

Finally, in relation with the planning of the future traffic the issue of delimiting the extent of the study should be mentioned. Many road projects only result in a local change within a very large network. Therefore, it is often a reasonable approximation only to analyse the effects of the traffic changes in the area where the project is carried out. Railway projects can typically be characterised as major changes within a small network, and major derived effects can therefore occur outside the area, where the infrastructure is changed.

The capacity conditions of the railway also mean that changes in the traffic on one subline can have big impact on the traffic on other line segments. The planner must therefore decide the size of the area where the traffic is to be analysed in order to evaluate the project. An extension of the line Copenhagen H – Høje Taastrup – Ringsted will for instance influence the railway traffic in the entire country, but nonetheless some analyses are restricted to local effects within the project area. The resources for consequence calculation will be an essential factor in this weighing.

1.5 The utilization problem

In this section a fundamental problem when evaluating railway projects, which here is called “the utilization problem”, is described. The issue is introduced, but due to its complexibility it is not within the scope of the introduction to railway traffic to go into depth with the subject. The concept of the utilization problems means that it is uncertain how the infrastructure will be used when the project has been finished (several years after the time of planning/evaluation).

It appears from the previous sections that the future timetable is partly unknown. Information about the frequency, the most important
connections and some probable rolling stock types may be available, but fundamentally the timetable is not known at the same level of detailing as the day’s traffic. However, the uncertainty is not the same on all lines. The timetable structure of the suburban railway has been almost unaltered from 1989 to 2006, whereas the timetable of the Copenhagen-Ringsted line practically changes each year. The uncertainty with respect to the project evaluation of the double track line Ballerup-Frederikssund has therefore been smaller than for the Copenhagen-Ringsted project.

It is not the intention to try to define precisely what “another” timetable means. In principle it is in the eyes of the travellers another/a new timetable, if some departures are moved a few minutes. But in relation to consequence calculation and partly capacity evaluation a few minutes sliding at a number of stations does not mean that you talk about “another” timetable (however, with respect to the regularity, the difference can be rather big!). As long as the order of the trains, connections and crossings is the same, it is in many ways the “same” timetable. In Dutch the word timetable structure is used for this level of description.

However, it is a fundamental problem in railway evaluations that the use of the infrastructure depends on the actual timetable. A classical example of this is the double track line between Holbæk and Vipperød. In the mid 80′es a double track line was constructed on this approximately 10 km long line, whereas the rest of the line Lejre-Holbæk-Kalundborg remained a single track line. At the time when the project was planned, the trains crossed in Holbæk, and the double track could therefore contribute to improved regularity.

However, at the time when the project was completed, the timetable of the line was changed so that the crossing does not take place on the same place any longer. This is due to the fact that the traffic on the Kalundborg line is not planned independently of the remaining network, and changes on the line Copenhagen H-Roskilde have resulted in derived changes. So the small piece of double track did not result in the benefits of punctuality that were expected when the project was planned. Even though the capacity conditions on single track lines are very special, it is also the case in other connections that the use of the infrastructure depends on the actual timetable. And consequently if the project is actually used as planned when it is opened to traffic.

During a socio-economic calculation period of (typically) 30 years timetable changes will of course occur, and it will hardly be reasonable

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13 It should be noted that there have been unusually many big changes in Denmark during the last 15 years due to the opening of the fixed links across the Great Belt and Øresund, transfer IC trains drawn by locomotives to train units and the electrification project. In Europe, so many big changes are unusual, both because they have not only implied timetable changes, but also changes in the frequency.
to integrate these in a cost-benefit analysis. The analysis must be based on an expected operation pattern/timetable when the project has been finalized. The issue is well-known, but it is difficult to suggest solution methods. If you imagine a re-location project in East Jutland evaluated on the basis of traffic prerequisites from 1976 – only 25 years ago – the result would probably be different if the evaluation was carried out in 1997. This thought experiment indicates that a cost-benefit analysis has its limitations as a planning tool for railway network extensions, among other things due to the long lifetime of the railway infrastructure.

The utilization problem thus means that it can be difficult to follow-up on the infrastructure investments – combined with the long period from planning to opening to traffic. Some of the operation pattern prerequisites may have changed, because there have been changes on other lines.

The utilization problem also means that it is difficult to generalize the effects of a railway project. It will depend on the timetable in question, and it is thus a fundamental problem in case of railway traffic. There have – especially within the capacity literature – been made many attempts to find universal descriptions of the infrastructure utilization. But they are mainly indicators used for dimensioning the infrastructure and not an exact description of the capacity utilization of a timetable.

The utilization problem and the dependency on timetables is a difficult issue in connection with railway planning. It is possible to work out candidate timetables showing that a project has big effects and candidate timetables showing that the project does not have any effects. A good example of this is the need for convergings out of grade between two double track lines. Normally, candidate timetables can be worked out in such a way that the project is necessary (crossing conflict) and unnecessary (no crossing conflict), respectively and candidate timetable method can therefore be used to show the conclusion you want.

It is not the intention to offer a solution to this fundamental problem relating to railway traffic, but it is relevant in order to understand the conclusions made on the basis of the consequence calculations of railway projects.

1.6 Summary

This chapter has provided an overview of the railway traffic and the most relevant conditions in relation to consequence calculations and project evaluations of railway projects. Emphasis has been placed on describing the differences with respect to road traffic, but primarily the differences between railway traffic and bus traffic. Even though there are similarities, the differences are however so big that the consequence calculations have to be done in different ways. The
Characteristics of railway traffic

The primary differences between buses and trains is that railway capacity has a substantial influence on the travel time, and the correlation between number of passengers and bus frequency is more simple than the equivalent correlation in case of railway traffic.

It is difficult to generalize consequence calculations for railway projects, because different lines have different mixes of train services, and because the timetable as a description of the actual capacity conditions has such a big influence on the project.

References

Chapter 2: Capacity conditions

All transport systems are characterised by consisting of a well-defined “infrastructure”, some transport units and a set of “rules of game”. When these 3 concepts are interacting, we talk about traffic. Each transport mode has its own characteristics, both with regard to infrastructure, transport mode and the “rules of games”. Since the interaction between these 3 concepts is of fundamental importance for the capacity conditions, the different transport modes will have different properties in this field.

It is common to all transport modes that the capacity $K$ in a cross section of the infrastructure can be described as:

Formula 2.1  \[ K = q_{\text{max}} \cdot n \]

Where: $q_{\text{max}}$ is the maximum traffic intensity e.g. [trains/h]  
$n$ is the number of train paths [-]

To be able to make a direct comparison of the performance of the individual means of transport, the capacity should be converted to number of passengers transported, which can be done in the following way:

Formula 2.2  \[ K_{\text{pass}} = N \cdot q_{\text{max}} \cdot n \]

Where: $N$ is the passenger capacity of the transportation unit e.g. [pass./train]

No matter the kind of transport mode, it will in case of a capacity analysis be natural to divide the infrastructure into, as a minimum:

- Lines (main line, lanes, corridors)
- Junctions (stations, road junctions/roundabouts, airports)

The lines are characterised by the fact that the individual transport unit will not meet crossing and opposite traffic. Therefore a rather big traffic intensity can be obtained in this case, since a high average speed $\bar{v}$ can be permitted due to a relatively smooth operation.
Capacity conditions

Formula 2.3 \[ q = D \cdot \bar{v} \]

Where: \( D \) is the traffic density\(^1\) e.g. [train/km]

As it appears from the above expression, the traffic intensity \( q \) does not only depend on the average speed \( \bar{v} \), but also on the actual traffic density \( D \). The maximum traffic density \( D_{\text{max}} \) will depend on the speed \( v \), driving behaviours (e.g. braking) and safety systems which determine the way the transport units can be operated. At high speeds the traffic density will thus be reduced as the braking distance requires a long overlap to the transport unit situated just ahead. The optimum speed \( v_{\text{opt}} \) will thus not be identical to the maximum speed \( v_{\text{max}} \), as it appears from the below figure.

Max. traffic intensity \( q_{\text{max}} \) [units/h]

![Graph showing maximum traffic intensity \( q_{\text{max}} \) as a function of the speed \( v \)[1]]

Figure 2.1 Maximum traffic intensity \( q_{\text{max}} \) as a function of the speed \( v \)[1]

It will often be more complicated to analyse the traffic flows in junctions than on lines. The traffic in junctions will often be characterised by certain traffic flows having to wait for the continued operation, or they must be operated at reduced speed. As stops have been introduced at junctions due to the passengers for certain means of transport, the traffic intensity \( q \) will often be smaller than on an open line. The capacity \( K \) will therefore be smaller in junctions than on free lines, unless this is sufficiently compensated by means of extra channels \( n \) (cf. formula 2.1) in the junctions.

\(^1\) The traffic density \( D \) is normally called headway distance \( S_t \) in case of railway traffic.
The safety is of great importance as to which capacity \( K \) can be obtained in connection with transport. This is the case because the safety issue will often determine how short a distance you can allow between the individual transport units, and it will therefore limit the traffic intensity. The technical systems underlying the safety influence the capacity. Long response times for permission to continuous operation and/or an inaccurate localisation of the individual transport units will thus reduce the capacity. The manoeuvrability of the means of transport will also influence the capacity of the system, as bad braking properties for instance result in a reduced maximum traffic density \( D_{\text{max}} \).

### 2.1 Railways

Train operation is characterised by consisting of a large number of bindings and dependencies. This is in itself an important capacity hindering factor. The infrastructure is thus very “inflexible”, i.e. overtaking can normally only take place at stations, and on tracks with bidirectional traffic crossings can only take place at the crossing stations.

Due to all operation being timetabled, the timetable should not be deviated, since it has normally been the intention to optimize the capacity with respect to passenger wishes. A delay will therefore often spread to other trains. When planning it is therefore not realistic to aim at obtaining a utilization ratio for the infrastructure of more than \( 60\% - 75\% \)\(^2\) of what is theoretically possible [9], as it is necessary to introduce buffer times \( t_b \) in the timetable which makes it possible for the system to regenerate.

The relatively high safety on railways can partly be ascribed to the very restrictive rules that railway traffic is subject to [3]. The safety system is normally designed in such a way that an error will not automatically lead to a serious accident. This is among other things done by introducing overlaps. At stations this is obtained by requesting a train path after the stop signal that will not be in conflict with previously set train routes. On an open track, the signal giving access to the next block section is placed one safety distance \( F \) before the block begins. The consequences of human errors, e.g. passing a red signal, will therefore not necessarily result in a dangerous situation, but it will limit the damage. Certain types of accidents can however be difficult to avoid. A modern ATC\(^3\) system will e.g. not be able to prevent a derailment from giving rise to a very serious accident, if the train due to the derailment invades another track where a train is approaching.

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\(^2\) On suburban lines, however, the utilization ratio can be higher

\(^3\) Automatic Train Control
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On most railway lines, the safety systems can still not determine the position and speed of the trains more precisely than the specific block section. This will typically be in the range of 1.5 km - 3 km. It means that the trains are only updated approximately once per minute, and the train therefore, in principle, occupies the individual blocks approximately 1 minute longer than strictly necessary. This, together with the demand for sufficient braking distance and overlap and a robust and customer friendly timetable, means that there can typically be obtained a practical headway time (= maximum traffic density $D_{\text{max}}$) of approximately 3 – 4 min/train, provided the traffic on the line is one-directional, and that the line is operated with several different train classes.

It is however possible to obtain much smaller headway times, particularly on city lines (suburban trains, subways or metros), where there is only one type of traffic on the infrastructure and where the technical safety systems are far more advanced. The Copenhagen suburban network more or less belongs to this category, at least with regard to the central section “the tube” (Copenhagen H - Østerport), with 30 timetabled trains/h in each direction in the peak hours which is equivalent to a practical headway time of 2,0 min/train [2]. In other big European towns, where it is not necessary to take into account punctuality troubles due to converging to different railway lines, a practical headway time as low as approximately 1.1 min/train (e.g. in Moscow and Leningrad) can be obtained.

The operation patterns of the Ørestadsbane (the Copenhagen metro) make it possible to obtain practical headway times in the rush hours of 1.5 min/train.

If the traffic intensity $q_{\text{max}}$ is converted by means of formula 2.2 to a passenger capacity $K_{\text{pass}}$, it will with the new suburban trains on the Copenhagen suburban network be possible to obtain the following passenger capacity on a track:

$$K_{\text{pass}} = 800 \text{pass/train} \cdot 30 \text{train/h} = 24,000 \text{pass/h}$$

It is assumed that there is only standing room available for 1/6 of the passengers in the approximately 180 m long trains.

If a similar calculation is made for the Ørestadsbane (the metro), it will despite of a smaller headway time not be possible to obtain a similar passenger capacity, as these trains are only 40 m long and has about 50 % less passenger capacity meaning that each trains can only hold approx. 180 passengers [8]. Therefore, it will only be possible to obtain 7,200 pass/h on each track.
On Châtelet in Paris up to 27,500 pass/h have been registered on each track [5]. Despite the shorter trains in urban railway traffic, it will typically be here that the biggest passenger flows can be obtained because of the very small headway times. On long distance lines there will seldom be more than 20 trains/h, so even though all these trains transport 1,000 passengers each (distributed on approximately 300 m), it is not possible to obtain passenger flows of the size found in urban railway traffic.

2.2 Road traffic

Generally, road traffic can be described as a traffic system that primarily consists of individual travellers that can travel on a very ramified infrastructure. The travellers’ behaviour is characterised by the fact that they obey to some overall rules (e.g. traffic regulations), but that the individual traveller chooses the route based on criteria such as passability, objective of the trip, travel time, etc. In case of bottlenecks in the system, some travellers will relatively soon choose an alternative route/travel time, especially in case of recurrent bottlenecks (same time and place).

The transport units is not as tightly fixed to the infrastructure in road traffic as in railway traffic. This means that the traffic can be operated more flexibly (i.e. overtaking and “crossings” are possible in many places), and at the same time a considerably denser flow of transport units “is allowed”. This is not only due to the flexible infrastructure allowing evasive actions, but also that the individual traveller determines the distance to the transport unit ahead of him, whereas railway traffic is subject to very restrictive safety systems that determine this distance.

Thus, the occurrence of accidents, measured as km travelled per person, is also approximately 5 times as big in road traffic as compared to railway traffic. In fact, the real difference is bigger, as a substantial part of the casualties and deaths in connection with railway traffic is due to conscious acts (suicides, sabotage and the like). Tragic as it may seem, the relatively poor safety in road traffic increases the capacity!!! - the high accident frequency can i.e. be interpreted as a result of the drivers taking many chances, e.g. dangerous overtaking and too short safety distances to the vehicles in front, which contributes to a faster flow of traffic (provided it does not lead to accidents!!).

In road traffic there is not a safety system that automatically ensures that the braking distance does not exceed the distance to the vehicle in front. When cars are driving bumper by bumper, the reaction length summed with the braking distance will normally be longer than the distance between 2 vehicles. This is not dangerous as such, as the vehicle in front will not suddenly stop, and evasive actions (e.g. change of lane) may be an option. However, such situations will never
Capacity conditions

occur in railway traffic, as evasive actions will never be achieved on the open line, and the danger point (= the train in front) is assumed to be stationary at the moment, where the necessary headway distance is determined.

In the below table, the expected capacity under the following ideal conditions is stated [1]:

- The width of the lane\(^4\) is 3,50 m
- The traffic consists exclusively of person cars
- The traffic is evenly distributed on the two directions of travel

<table>
<thead>
<tr>
<th>Lane type</th>
<th>Road capacity [cars/h]</th>
<th>Road capacity [pers/h](^5)</th>
<th>Lane capacity [cars/h]</th>
<th>Lane capacity [pers/h](^5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-lanes</td>
<td>2.200</td>
<td>3.300</td>
<td>1.100</td>
<td>1.650</td>
</tr>
<tr>
<td>3-lanes</td>
<td>4.000</td>
<td>6.000</td>
<td>1.333</td>
<td>2.000</td>
</tr>
<tr>
<td>4-lanes</td>
<td>8.000</td>
<td>12.000</td>
<td>2.000</td>
<td>3.000</td>
</tr>
<tr>
<td>6-lanes</td>
<td>12.000</td>
<td>18.000</td>
<td>2.000</td>
<td>3.000</td>
</tr>
</tbody>
</table>

Table 2.1 Capacity on roads [1].

It is seen that many lanes give a higher average capacity per lane. This is due to the increased possibility of overtakings and passings. It thus becomes easier to separate fast and slow travellers. That is because homogeneous traffic flows are a prerequisite to obtain optimum capacity utilization in a train path.

If the maximum passenger flow per train path (= lane) in the above table are compared with the capacity of railway, it can be observed that a railway track is capable of operating a passenger flow that is approximately 7 – 9 times bigger than what can be obtained on a road lane.

\(^4\) In the case of bigger lane width, bigger capacities can be obtained as the possibility of overtaking and evasive actions is increased.

\(^5\) The conversion from [cars/h] to [pers/h] is base don the assumption that there on average are 1,5 persons in each car.
2.3 Air transport

Just as railway traffic, air traffic can be characterized as public transportation controlled by the central control centres\(^6\).

One might think that space has “unlimited” capacity. This is however not the case, as airplanes have to use predetermined corridors (in several layers) between the airports.

These corridors (= airways) are approximately 18 km broad and 10 km high paths, where the vertical distance between the individual planes must be 0,6 km [6]. Due to weather conditions and bottlenecks in the terminal areas (i.e. close to the airports), it will however not be possible to operate in more than 4 – 6 levels in each direction of a corridor [4].

Each level of the corridor only allows traffic in one direction, as a safety zone of 9 km on each side of the plane is required. The distance between 2 planes flying right behind each other must be at least 8 km when using radar [7]. Even though several planes today have maximum speeds of approximately 1.000 km/h, the average operation speed in the different layers will be in the range of 500 km/h, as both weather conditions and slow planes will limit the speed. The practical attainable capacity for one direction in an aerial corridor can therefore be determined to:

\[
K = \frac{6 \cdot 500\text{km/h}}{8\text{km/plane}} = 375\text{planes/h}
\]

An aerial corridor can thus operate considerably more transport units than a railway track, but if the capacity is calculated per layer, the difference is not very big.

In case of air traffic it is however normally the terminal areas that limit the capacity. As opposed to railway traffic, it is difficult to establish a sufficient number of paths in the junctions. At large central stations outside Denmark, it is not unusual with 10 to 20 platform tracks, whereas the maximum number of takeoff runways and runways in airports is 4 due to safety reasons. In case of railway traffic, one extra platform track per track on the open line will normally prevent the station from limiting the capacity, as it will allow for overtaking and stops without troubling the traffic in the other direction. In case of air

\(^6\) However, small planes may base themselves on the rules of flying visually/visual flight provided the visibility is 8 km and the vertical distance to the clouds is at least 300 m [6]. In this chapter, however, only flying based on rules of instrument flight is described.
Capacity conditions

transport, the number of paths will however always be reduced in the transition from air corridors to terminal areas. Around the terminal area, there will thus be some 33 km times: 43 km big waiting areas [6], where the planes can circulate until they get permission to land. In connection with take-offs and landings, the mutual location of the takeoff runways and the runways is of great importance with respect to the capacity. Under optimum conditions, 2 parallel lanes could operate 80 landings per hours or 120 takeoffs per hour. The average lane capacity is therefore 50 planes/h. Under the assumption of approximately 200 pass/plane, the practical passenger capacity $K_{\text{pass}}$ of a runway in the airport could be calculated to:

\[ K_{\text{pass}} = 50 \text{planes/h} \cdot 200 \text{pass/plane} = 10.000 \text{pass/h} \]

Apparently, the passenger capacity of a path in case of air transport seems somewhat smaller than that of railway operation. But if the Copenhagen city railway is used as an example, it would not be possible to maintain an operation with 30 trains/h in each direction on double track on the open line, if the central station of Copenhagen Central (København H) did not have 4 platform tracks reserved for the city railway which makes it possible for the traffic to regenerate. Furthermore, the passenger capacity $K_{\text{pass}}$ of takeoff runways and runways will of course be somewhat bigger, if they are only operated with large planes, each with a passenger capacity of approximately 300 pass/plane. Normally, the size of the planes differs significantly, so the phenomenon similar to city railways, where all transport units are identical when the traffic flow is big, will seldom be found in air transport.

It can therefore be concluded that the passenger capacity per train path in junctions is somewhat bigger in case of railway transport as compared to air transport, whereas the opposite is the case when it comes to the lines between the junctions.

References
Capacity conditions
Chapter 3: The Capacity Concept

The word capacity means to yield/carry. Therefore, capacity can be defined as the ability to transport freight and passengers on railways.

In the railway sector, capacity is mentioned in many different connections. The word can be used to describe the capacity of a train (e.g. the number of passengers or tonnes that the train can carry, the capacity of a line or a station or the capacity of the entire network (e.g. the number of trains the network can handle [trains/h]).

According to the laws of mechanics, when talking about traffic, capacity is often defined as volume multiplied by distance per time unit. In physics, capacity is thus defined as the work produced during a time unit [Nm/s]. This also applies to railway operation facilities, where “work” can be defined as transport units through an operations facilities. If the “volume” consists of travellers or goods, you talk about traffic capacity. It is persons multiplied by distance per time unit (e.g. [pkm/day]) or tonnage carried multiplied by time unit (e.g. [tkm/day]). However, if the “volume” consists of transport units (e.g. trains or wagons), you talk about transportation capacity. The transportation capacity is the transport units multiplied by distance per time unit (e.g. [train-km/day]). Whether the transport units are empty or full does not influence the capacity of the operation facilities.

The capacity of an infrastructure does not only depend of its design, but also on the properties of the trains that have to use it. Furthermore, the order in which the different train classes are operated on the infrastructure also has a major impact on the capacity. The capacity of a line can thus change without the infrastructural conditions being changed. Therefore, a plan of operation will not be sufficient to describe the exact capacity. It requires an actual timetable.

Capacity can be described at the following 3 levels:

- The operators’ required capacity (capacity demand) based on prognoses (cf. section 3.1)
- Technically possible capacity¹ (cf. section 3.2)
- Capacity rendered at the actual capacity conditions (cf. section 3.3)

3.1 Capacity demand and flow

A capacity demand can be described by the following 3 characteristics:

- Requirement
  - The number of trains required during a given period on a given infrastructure, e.g. [trains/h].

¹ Does not include restrictions imposed by environmental and financial conditions
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- Sequence and train characteristics
  - The individual order of the trains\(^2\) and the properties of the transport units must be described to examine the possible capacity on a given infrastructure.

- Frequency
  - The time distances between the transport units (operation plan) or number of transport units in defined time intervals during the observation period (plan of operation)

The capacity demands will be made on the basis of a number of wishes from the various operators to the new timetable. In the light of the future liberalization of the right to operate on the public railway infrastructure, the number of operators will probably increase considerably. Until ten years ago, the use of the Danish infrastructure has been reserved to DSB through the different operators (Freight, International, InterCity, Regional trains and the suburban railway). At an early stage, the operators’ wishes are entered into a plan of operation that must express the level of service they would like in the future.

3.2 Possible capacity

The possible capacity of an operations facility is its ability to obtain a certain capacity under the assumption of an unlimited capacity demand\(^3\) with a given operational structure (e.g. train mix).

The structure of the capacity demand (order and transport unit properties) is important when studying the possible capacity. Changes in the structure of the capacity demand thus lead to different possible capacities.

The lower possible capacity compared to the maximum capacity is not due to less requirements to the operations facilities, but only as a

\(^2\) For instance, fast and slow trains, respectively, can drive in larger pools (so-called bundling) to increase the capacity.

\(^3\) When the capacity of the capacity demand \(K_{\text{max}}\) (cf. section 3.2.1) has been obtained, there are not buffer times between the trains which are necessary to avoid transmission of small delays to other trains. Randomly, but with a probability which can be determined, there will occur delays at the beginning and in connection with the implementation of the individual actions which consequently leads to delays of the subsequent actions. If no "periods for regeneration" have been planned, and the system permanently behaves in the above way, there will be an increasing queue of waiting transportation units in the system which will result in constantly increasing waiting times. Thus, the system will not be in statistical balance.
result of less reliability and accessibility to the infrastructure, rolling stock and crew.

By definition you, therefore, often distinguish between the theoretical (maximum) capacity $K_{\text{max}}$ (cf. section 3.2.1), the actual (fundamental) capacity $K_f$ (cf. section 3.2.2) and the actual (available) capacity $K_a$ (cf. section 3.2.3).

### 3.2.1 Maximum capacity $K_{\text{max}}$

With regard to the theoretical maximum capacity $K_{\text{max}}$, it is assumed that the trains run in an optimum order with the smallest possible distance allowed by the signal and the safety systems, however in such a way that the trains can operate at maximum speed. In this way, there are no buffer times $t_b$ that can absorb the delays, and almost all delays will therefore spread to several other trains (in worst case to all of them). Thus, the maximum capacity $K_{\text{max}}$ describes the ability of a system to obtain a certain capacity based on its technical equipment (signals, wiggly wires, etc.) and the infrastructure. It is a prerequisite that the capacity demand is unlimited, and that all necessary rolling stock and crew are available. Another prerequisite is a smooth operation. The maximum capacity $K_{\text{max}}$ can only be realized over a short period of time, as it will be difficult to operate more-trains on a line with such a big flow. Thus, maximum capacity $K_{\text{max}}$ can only be obtained on a line when the headway time $t_{h,\text{min}}$ is as small as possible and without buffer time $t_b$. There will therefore not be any time for regeneration, if a line is utilized equivalent to $K_{\text{max}}$.

### 3.2.2 Fundamental capacity $K_f$

The fundamental capacity $K_f$ is normally smaller than the maximum capacity $K_{\text{max}}$. It allows for restrictions in the reliability of infrastructure, rolling stock and crew.

These restrictions are permanent and can be evaluated. They occur randomly (can be described as statistical) or as planned and recurrent (e.g. preventive maintenance of the infrastructure).

Depending on the probability of failures, the fundamental capacity $K_f$ can adopt different values. However, there is no method which can be used to calculate it explicitly based on the maximum capacity $K_{\text{max}}$.

On the other hand, the theoretical capacity $K_{\text{max}}$ can normally be quantified relatively exactly. Due to the lack of explicit calculation rules, the practical attainable capacity $K_f$ will often be assessed as a percentage of the theoretical capacity $K_{\text{max}}$.

This assessment will rely on data experience regarding the quality $Q$ of the actual operations carried out. For more profound studies, it will also be relevant to use simulation models to reproduce the expected operation. If nothing else is known, according to Code UIC 405 R [3]...
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you may assume that you on a daily basis can set \( K_f = 60\% \cdot K_{\text{max}} \), whereas you in a single maximum hour can obtain \( K_f = 75\% \cdot K_{\text{max}} \).

Regarding the concept of practical attainable capacity \( K_f \), it is assumed that the traffic must be of a certain quality, in this case punctuality. Therefore, there must only be a certain maximum of delays (troubles). If the delays are unacceptably big, it indicates that the practical attainable capacity \( K_f \) of the infrastructure is exceeded. The definition of practical attainable capacity \( K_f \) can, therefore, be described in the following way:

\[
\text{The capacity of an infrastructure facility is the ability to operate the trains with an acceptable punctuality.}
\]

This definition is not operational as it requires a further definition of the concept of acceptable punctuality. The infrastructure can be utilized beyond its practical attainable capacity \( K_f \), but not beyond its theoretical capacity \( K_{\text{max}} \), as it is based on full utilization of running times, safety rules, etc.

3.2.3 Available capacity \( K_a \).

In case of a prolonged shortage on facilities, rolling stock and/or crew, there is a reduced capacity \( K_a \) available as compared to the fundamental capacity \( K_f \).

All capacity indications must refer to periods where processes can be considered stationary, i.e. that the mean value is characterized by “the structure” of the capacity demand as well as infrastructure, rolling stock and crew, is constant, and the variance is low.

Depending on the different structures of the capacity demand, there are also different capacities with the same composition of infrastructure, rolling stock and crew. The correlations are shown in figure 3.1.
Figure 3.1 Example of the influence of the structure of the capacity demand on the capacity.

Figure 3.1 shows that 2 different timetable constructions (structures 1 and 2) consisting of the same trains, but combined differently, can give rise to different capacity $K$.

### 3.3 Capacity conditions

The capacity conditions of a system describe the functional correlation between the traffic load $A$ and the quality $Q$ depending on the capacity $K$, amount of transport units and their distribution in time (plan of operation). Whereas the capacity is always examined under the assumption that the capacity demand is unlimited during the reference period, the influences from the inhomogeneous rush of passengers and the quality obtained are taken into consideration when studying the capacity conditions.

#### 3.3.1 Quality

Quality is a very broad concept. In connection with capacity studies the concept of quality is concentrated on “operation quality” with the components “timetable quality” and “the quality of the operations carried out”. Within these, with the characteristics of “speed” and “fulfilment of the wanted departure times/stopping patterns” (wishes to the timetable) and “punctuality” (operation carried out).
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operation quality depends on the traffic load $A$ of the infrastructure. The intensity includes all the characteristics of a capacity demand. The planned and unexpected waiting times and delays are suitable for measuring the quality. Thus, the operation quality declines with increasing waiting times and delays.

The expression delays is characterized by part of these being the queuing time that a given infrastructure imposes on the individual, because there are several trains in the system. They can also be described as secondary delays that are characterised by not relying on randomness, but on the intensity. However, primary delays are initial delays (at the departure station), signal problems, accidents and the like which are not directly depending on the infrastructure capacity. On the other hand, the infrastructure capacity is essential for the way in which the primary delays will spread (as secondary delays) to the other trains in the system.

The following presentation is based on queuing time as the quality criterium. The correlations are only presented qualitatively, i.e. each combination of infrastructure and intensity results in a special formula for the queuing time function.

![Figure 3.2 Queuing time $t_q$/quality $Q$ as a function of the traffic load $A$.](image)
Figure 3.2 shows the functional correlation between quality (queuing time) and traffic load. The higher the traffic load, the bigger the queuing time. If the intensity on an operations facilities is close to the max capacity limit, the queuing time will be extremely high. Based on the queuing time function it is seen very clearly that each utilization value is related to a certain objective for the attainable operation quality, and that the traffic load A and the quality Q cannot be maximised at the same time. A prerequisite for obtaining quality is the existence of a buffer time $t_b$. The size of the queuing time function varies depending on the level at which the capacity of an operations facility is defined.

In figure 3.3 the lower curve thus expresses that the operation is based on an optimum utilization of the infrastructure (equivalent to $K_{\text{max}}$ at the biggest possible load). On the other hand, the intermediate curve is based on an operation that is equivalent to a realistic timetable (i.e. the biggest possible load is $K_f$). Finally, the upper curve shows the conditions in connection with an actual operation (here the maximum intensity is thus equivalent to $K_a$), where temporary restrictions of infrastructure, infrastructure and/or crew may occur (cf. section 3.2.3).

![Figure 3.3 The queuing time function at different traffic loads.](image)
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The exact shape of the curve will however be very dependent on the construction of the infrastructure in question.

The queuing time function can also, depending on the distribution in time of the load, adopt different forms at the same capacity. The impacts on the uniformity of the plans of operation are shown in figure 3.4.

![Diagram showing queuing time function in case of different operation plans.](image)

Homogeneity of the plan of operation

Figure 3.4 The queuing time function in case of different operation plans.

Curve I is thus an expression of an operation plan (= timetable) with a very bad grouping of trains with similar operation pattern with respect to the order in which the infrastructure is used. In this way, the queuing time is increased relatively fast in case of an increased intensity. However, curve IV is the result of an operation plan where trains with the same operation pattern occur in pools when using the infrastructure.
3.3.2 The optimal field of capacity

The optimal field of capacity of an operation facility is the one where the attainable benefits are bigger than the necessary costs that lead to capacity. The optimal capacity point is the one where the difference between the benefits and the costs is biggest. The costs, which depend on the load, are shown in figure 3.5. Figure 3.5 shows a strongly progressive shape for the costs as a function of the load. The progressive shape is due to the fact that the queuing time which can be estimated (time is money).

![Cost function diagram](image)

Figure 3.5 The cost function.

The total operational costs $C_0$ can thus be expressed as the sum of the fixed costs $C_{fix}$ (not directly related to the operation) and the costs directly related to the operation. In practice, it is difficult to distinguish between the fixed and variable operational costs, as these amounts,
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when it comes to infrastructure, crew and rolling stock, can only be adapted over a longer period of time in case of major operational changes. This means that the fixed costs should be estimated relatively high as compared to the variable costs. However, in case of minor operational changes, which do not change the total system significantly, it will normally be sufficient to carry out internal redistributions, especially with respect to rolling stock and crew. In this way the costs will appear more as variable costs if you look at a small sub system. If the total operational costs are added to the extra running time $t_q$ (cf. figure 3.4) which the passengers are imposed in a very congested system, you obtain an expression of the social costs $C_s$. This is done based on the motto that time is money.

However, not only the costs will increase with increasing utilization, but because of the increasing queuing time the speed at which the transport units can run on the infrastructure will also be reduced. A consequence of this is, that it may be necessary to buy more rolling stock and crew, as the increased turnaround time can cause that the unit (train/crew) is not available the next time a departure is wanted.
The shape of the cost function $C_{\text{total}}$ can with good approximation be described as a construction oriented cost calculation (supplemented with the queuing time factor). On the other hand it is somewhat more difficult to describe the benefit function $B$ (Benefits). It is because the time-dependent elasticity of demand shows a big span with respect to line segment and season and product dependencies. Therefore, it is not possible to make a clear and “correct” benefit function of an infrastructure facility, unless you set up a large number of actual prerequisites.
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Figure 3.7 The queuing time \( t_q \) and its relative sensibility \( t_q'/t_q \) as a function of the traffic load \( A \).

If you measure the increase in the queuing time function \( t_q \) at a certain traffic load, in other words the load in the growth of the queuing time, and propose this value to the queuing time \( t_q \) at the actual load, you obtain the relative sensibility of the queuing times relative sensibility at this intensity. The shape of this function is shown in figure 3.7. Thus, the size \( t_q'/t_q \) is thus an expression of the relative sensibility of the queuing time\(^4\). Or in other words, the relative growth factor of the queuing time in case of an increasing traffic load seen in relation to the actual queuing time. The shape of this curve thus shows when the queuing time tends to “explode”.

If the queuing time, the minimum running time, and the necessary dwell time is measured in a system, it will be possible to express the speed in the system as a function of the intensity. Due to the increasing queuing time at increasing loads, the average speed \( v \) will

\[ \text{t}_q' = \frac{1}{A (1 - A)} \]

\(^4\) Professor Hertel from Dresden University has used the relative sensitivity of the queuing time in connection with the description of waiting time systems. If it is assumed that the arrival distribution of the trains follows an exponential distribution and that the service time follows a random distribution, the relative sensitivity of the queuing time will be equal to \( 1/(A(1-A)) \) as shown in figure 3.7.
therefore be reduced similarly. The function approaches 0, when the intensity approaches the capacity level \( K \) which appears from figure 3.8.

\[
\text{Speed } v \quad \text{Traffic energy } E
\]

\[
\text{Queuing time } t_q
\]

\[
\text{Max } v \cdot E
\]

\[
\text{Max } t_q
\]

\[
\text{Traffic load } A
\]

As the capacity and the quality (in the shape of average speed) cannot be maximized at the same time, but may even present the opposite shape, it will be natural to maximize the product of the traffic load \( A \) and the speed \( v \). Apart from indicating the capacity \( K \) of an operations facilities it will therefore also be interesting to find out at which speed \( v \) this capacity can be obtained.

The product of capacity \( K \) (average) and the speed \( v \) in the system can thus be defined as the traffic energy\(^5\) \( E \). This is also shown in figure 3.8 as a function of the traffic load \( A \). The traffic energy \( E \) has a maximum and approaches 0 for maximum traffic load \( A \), as the permanent

\(^5\) If you calculate the product of the capacity \([\text{volume} \cdot \text{distance} / \text{time}]\) and the speed \([\text{distance} / \text{time}]\), a unit \([\text{volume} \cdot \text{distance}^2 / \text{time}^2]\), which is proportional with the kinetic energy \([\frac{1}{2} \text{ mass} \cdot \text{speed}^2]\) is achieved. Therefore, the product of capacity and speed is called traffic energy
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operation of a system with a traffic load $A$ equivalent to capacity $K$ leads to increasing queuing time (and consequently to the time used in the system). As the traffic energy $E$ contains terms like travel time and volume, it is a good expression of the benefits rendered by a system at different loads. The maximum value of $E$ describes a field of capacity that should not be exceeded, as it will lead to a negative influence from the strongly reduced speeds (and thereby also the traffic energy). If you describe a dependency for the demand as a function of the speed, the benefit function will thus react in the same way.

The optimal field of capacity is found by determining at which traffic load $A$, the extreme value of both traffic energy $E$ and the relative sensibility $t_q'/t_q$ of the queuing time occurs.

Studies from a number of German railway lines show that the relative queuing time sensibility $t_q'/t_q$ on the studied lines is smallest at a traffic load $A = [0.47 ; 0.54]$. 

Figure 3.9 Optimum traffic load $A$. 

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On the same lines, the traffic energy $E$ is also determined. For the studied lines, this function assume the maximum value at a traffic load $A = [0.75; 0.81]$. This traffic load is approximately equivalent to the maximum traffic load ($A = 75\%$) prescribed by the International Railway Union (UIC) as the general upper capacity level. The optimum capacity area must generally be expected to occur at a traffic load that lies in the range of $50\% – 75\%$, provided that the optimizing criteria include for instance high speed $v$, small queuing time $t_q$ and big traffic load $A$.

References

[1] Hertel, G. et al., The maximum operation and the minimum sensibility of the timetable on railway lines, Eisenbahn Techniche Rundschau, Volume 41, October 1992 (In German).


The Capacity Concept
Chapter 4: Determination of headway times

To determine the capacity of a given line/station you must know the “traffic regulations” and technical limitations of optimal driving. Depending on the technical systems used (both safety installations, train control systems and rolling stock), the occupation of a segment of the infrastructure will, give rise to a bigger or smaller blocking of the adjacent infrastructure. The dependence of the headway times on the technical equipment of the infrastructure, such as signal and safety systems, is discussed in sections 4.2 – 4.4, where the principle of calculation of different technical infrastructures is described in details.

The headway times will analogously with the service times of the queuing theory express how long time a train (a unit) occupies a specific part of the infrastructure (the system).

By minimum headway time $t_{h,\min}$ is meant the smallest distance in time that can be maintained between two trains, so that the trains behind can run at an optimal speed\(^1\). Or in other words: the smallest distance in time at which the trains behind can drive without being troubled by the trains in front. This distance in time must be calculated between the same fix points of the trains (normally the front). In the following, only the headway time $t_h$ [min/train] is described, even though minimum headway time $t_{h,\min}$ is the right term.

The minimum headway time can be obtained, if the line is operated with homogeneous traffic, i.e. all when the trains operate with the same speed and stopping pattern. The practical attainable Capacity $K_f$ [trains/h] can in these cases be determined to:

\[
K_f = u \cdot \frac{\Delta T}{t_b}
\]

Where: $\Delta T$ is the observation period, e.g. [60 min/h]

$u$ is the utilization ratio\(^2\) of the maximum capacity $K_{\text{max}}$ [-]

The practical capacity $K_f$ can also be expressed by means of the buffer time $t_b$ between the individual trains in the following way:

\(1\) The optimal speed is the most restrictive speed of the line speed and the maximum speed of the train. In connection with planned stops, the optimal speed will however be lower, as it will be a function of the retardation and acceleration properties and the distance between the halts.

\(2\) It can be set at 60% if examined daily, whereas the utilization ratio is 75%, when the maximum hour is examined [6].
Determination of headway times

Formula 4.2 \( K_r = \frac{\Delta T}{t_h + t_b} \)

By inserting formula 4.2 into formula 4.1, the buffer time \( t_b \) [min/train] can be expressed by means of the utilization ratio \( u \) and the headway time \( t_h \):

\[
 u \cdot \frac{\Delta T}{t_h} = \frac{\Delta T}{t_h + t_b} \iff t_b = \frac{t_h}{u} - t_h
\]

\[
 t_b = \left( \frac{1}{u} - 1 \right) \cdot t_h
\]

**4.1 Time elements in the headway time**

This section contains a detailed explanation of which time elements that form the basis of the headway time. The most restrictive technical system (cf. section 4.2), which only consists of balises, where the speed (and position) of the train is registered), as well as signals, where the permission to continue driving is given, is used as an example, as it is the one containing most time elements. To be able to maintain an optimum speed, all the signals met by the trains must be green. In Denmark, the signalling is designed in such a way that two green (GG) lamps in the signal means that the two next blocks will be free, and the train can therefore continue at optimum speed. Similarly, one green (G) lamp means that only the first block is free, and the train must normally start to brake gradually, as signal number 2 must be expected to be red, when the train reaches it.
Figure 4.1 shows an example of the signal picture behind a train in Denmark. At the place where the horizontal broken lines cut the vertical broken lines, the positions of the signal will thus change as indicated. For train 2 to be operated unimpededly, it must not come closer to train 1 than it can see the two green lamps before a possible gradual braking is to be started. In case of an ATC-braking, the gradual braking is both determined by the speed and the distance to the signal/danger point.

The time for setting up the train route is the time that passes from the moment that the safety installation registers that the blocks are free until the signal lamp turns green. This time is marginal, i.e. only a few seconds. Depending on the nature of the safety installation (mechanical interlocking system, free wired interlocking, geographical interlocking system, computer based interlocking system) there will however be minor variations. But even in case of a manual mechanical system, the time for setting up the train route will not take more than 10 seconds provided however that the station inspector is ready to carry out his job.

The signal realizing time is the time it takes the driver to watch the signal and know how to react. In Denmark, it must be at least 3 seconds [3], and as the driver is supposed to be unaware of the signal for up to 3 seconds, the total signal realizing time is therefore set at 6 seconds, but in practice it may feel shorter.

The time for braking of the train will of course depend on the braking properties of the train and the actual speed. In figure 4.2 the time for braking is put in inverted commas, as it should not be understood as the actual braking time. In figure 4.2, “the time for braking” will
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therefore be the time it takes to drive the distance between the caution signal and the block signal at the optimum speed. Consequently, “the time for braking” will be shorter than the actual braking time. In Denmark the presignal will typically be replaced by an ordinary signal, which however only means that the “time for braking” will normally be longer. On the other hand, it is not necessary to duplicate the block signals with presignals. With a block length of 2.000 m and an average speed of 120 km/h, "the time for braking“ will thus be one minute.

*The time in the block section* is the time it takes a point on the train (normally the front) to pass the block section. Therefore, it can be calculated in the same way as the “time for braking”, if no presignals are used. The calculation however has to be made in the following block section.

*The clearing time* is the time that passes before the entire train has left the blocks, as well as the following safety section\(^3\). In Denmark the safety section will typically be approx. 150 m on long distance railways and approx. 80 m on the Copenhagen suburban railway. Therefore, *The clearing time* will typically be in the range of 10 – 12 seconds for trains with a length of approx. 250 m and an actual speed of approx. 120 km/h.

*The release of a train route* is marginal just as the time for setting up the train route, as it only takes a few seconds to neutralize routes and adjust switches and signals.

The total *block occupation time* \(t_B\) will therefore be in the range of 2½ minutes, if a train of approximately 250 m drives through a block section of 2.000 m without presignals at a speed of approximately 120 km/h.

\(^3\)The safety section (or overlap) is the infrastructure where the block sections are in principle overlapping.
Figure 4.2 Time elements of the train sequence.

The block occupation time $t_B$ can be understood as the headway time $t_h$, if it is the dimensioning block sections on the open line. In fact, this is how professor Schwanhäußer from Aachen University determines headway times. This method thus focuses on the train’s occupation in time.
Determination of headway times

However, professor Hertel from Dresden University has another conception of headway time. According to figure 4.2 he observes the time between two trains’ passage of a fix point on the infrastructure (e.g. a block signal) as the starting point.

If the trains drive at a constant speed, the headway time will be the same whether you use professor Schwanhäußer’s or professor Hertel’s method, which appears from figure 4.2. However, if the speed is not constant, the headway time depends on the calculation method, as the physical “analysis area” is not identical.

For practical measuring practical headway times, Hertel’s method is the most suitable, if you place yourself by the track. However, if you have access to a visual control panel at the train control centre, it will be easier to use Schwanhäußer’s method to determine the block occupation times which are found by observing the time each of the diodes are lit.

4.2 Discreet block and discreet ATC

Figure 4.3 shows how you can determine the headway distance \( S_h \) [m], provided the trains operate based on track side signals (discreet block) and with discreet ATC updating (discreet ATC). Basically, the Danish ATC system is constructed in such a way that the train receives information discreetly from the balises. Therefore the below situation is very typical for a Danish railway line, as it is furthermore based on external signalling.

![Train sequence at discreet block and discreet ATC](image)

Figure 4.3 Train sequence at discreet block and discreet ATC [2].
As appears from figure 4.3, the following restrictions have to be fulfilled, if the trains are operated on lines with discreet ATC\(^4\) and discreet block sectioning\(^5\).

Formula 4.4  
\[
S_h \geq \sum_{i=1}^{n} B_i + L + S_s
\]

Formula 4.5  
\[
\sum_{i=2}^{n} B_i \geq S_b
\]

Where:  
- \( B_i \) is the length of the \( i \)’th block section [m]  
- \( i \) is the block section number behind train 1 [-]  
- \( n \) is the number of free block sections between train 1 and train 2 [-]  
- \( L \) is the length of train 1 [m]  
- \( S_s \) is the security distance\(^6\) (normally approximately 150 m in Denmark) [m]  
- \( S_b \) is the braking distance of train 2[m]

When the capacity of a railway line is to be determined, it is however the headway time \( t_h \) and not the headway distance \( S_h \) that gives the direct answer.

The headway time \( t_h \) is determined in the following way:

Formula 4.6  
\[
t_h = \frac{S_h}{v_2}
\]

Where:  
- \( v_2 \) is the speed of train 2 in the actual block section(s) [m/s]

\(^4\) Information about trains and free block sections is only updated when the trains pass the balises.  
\(^5\) I.e., the trains are operated based on external signalling along the track.  
\(^6\) The principles for indicating the overlap are somewhat different. In Denmark and England it is indicated as shown in figure 4.3, where the signal and the physical separation joint between the blocks are separated by the overlap \( F \). Therefore, train 1 is situated exactly where block section \( L_{b,1} \) is cleared. In Norway, the signal and the electric separation joint are coincident, and the braking distance \( S_b \) must thus be added to the overlap \( S_s \), when the block length is determined. [4]
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If train 2 is driving at too high speed $v_2$, so that the restriction in formula 4.5 cannot be met, train 2 lowers the speed in the following block section causing queuing time $t_q$ and thereby a bigger headway time $t_h$. The smallest headway time is obtained if:

- Train 1 and train 2 are operated at the same speed $v = v_1 = v_2$
- The headway distance $S_h$ is minimized with due respect to the restriction in formula 4.4

The trains are operated at the highest possible speed $v$, cf. formula 4.6

However, the two last points give rise to a conflict, as a small headway distance $S_h$ will lead to a small braking distance $S_b$, cf. the restriction in formula 4.5, which again leads to a low speed $v$, cf. formula 4.7. Therefore, the optimum is obtained by an iterative fulfilment of the criteria.

Formula 4.7

$$S_b = \frac{v^2}{2 \cdot r}$$

Where: $r$ is the retardation [m/s²]

Formula 4.7 is a simple description of the braking distance $S_b$ of the train. A more correct determination of the braking distance can be found by means of the below expression:

Formula 4.8

$$S_b = \frac{1}{2 \cdot (c \cdot r_b + r_g)} \cdot v^2 + (t_r + t_p) \cdot v$$ [5]

Where: $c$ is the braking ratio [-]

- $r_b$ is the braking retardation [m/s²]
- $r_g$ is the gravity retardation [m/s²]
- $t_r$ is the reaction time⁷ [s]
- $t_p$ is brake propagation time⁸ [s]

---

⁷ From the activation of the brake lever till the braking is initiated. For calculation purposes it can be assumed that $t_r = 3$ s [5]

⁸ Until a maximum braking effect has been obtained for the entire train. For calculation purposes it can be assumed that $t_p = 0.008$ s/m · L [5]
As appears from formula 4.8, the braking distance $S_b$ depends on several parameters. However, the gravity retardation $r_g$ can be ignored, if gradient $g$ of the line is very small ($-5\% \leq g \leq 5\%$), otherwise it is calculated in the following way:

Formula 4.9  \[ r_g = G \cdot g \]

Where:  
- $g$ is the gradient of the line (negative in case of down-grade)
- $G$ is the acceleration of gravity ($9.82 \, \text{m/s}^2$ in Denmark)

The braking retardation $r_C$ in equation (4.8) depends on the train class (and the length), which is determined by means of the Mindener formula:

Formula 4.10  \[ r_b = \frac{6.1 \cdot C + 61}{1.200} \, \text{m/s}^2 \]  

Where:  
- $C$ is the braking percentage [-]

It can for instance be mentioned that the braking percentage $C$ is 184 for IC3-trains, whereas it is 80 for freight trains ($L = 725 \, \text{m}$).

The last term in formula 4.8 is due to the reaction time for the brakes $t_{br} \, [s]$ that describes the prolongation of the braking distance which is partly due to the reaction time $t_R$ and partly to the brake propagation time $t_p$.

Formula 4.11  \[ t_{br} = t_R \cdot t_p \]

### 4.2.1 Example of calculation

If you have an IC3 train consisting of 5 train units (i.e. $L = 294 \, \text{m}$) that drives at a speed of $180 \, \text{km/h}$ ($v = 50 \, \text{m/s}$), you get the following braking distance $S_b$, cf. formula 4.8, on a line with a gradient of $10\%$ downwards ($g = -10\%$) provided the automatic train stopping device/ATC-automatic application of the brakes (in Denmark that means $c = 70\%$) is used:
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\[
S_b = \frac{1}{2 \cdot \left(70\% \cdot \frac{6.1 \cdot 184 + 61}{1.200} + 9.82 \cdot (-1\%) \right)} \cdot (50 \text{ m/s})^2 + (0.008 \text{ s/m} \cdot 294 \text{ m} + 3 \text{ s}) \cdot 50 \text{ m/s} = 2.379 \text{ m}
\]

If the block length is \( L_B = 2.000 \text{ m} \) on the entire line operated by the train, the necessary number of free block sections \( n \) can be determined using the restriction in formula 4.5:

For \( n = 2 \):

\[
\sum_{i=2}^{2} L_{B,i} \geq S_b \Rightarrow 2.000 \text{ m} \geq 2.379 \text{ m}
\]

For \( n = 3 \):

\[
\sum_{i=2}^{3} L_{B,i} \geq S_b \Rightarrow 2 \cdot 2.000 \text{ m} \geq 2.379 \text{ m}
\]

The restriction is thus not fulfilled for \( n = 2 \), but for \( n = 3 \). Formula (4.4) can therefore be used to determine the headway distance \( S_t \), for \( n = 3 \):

\[
S_h \geq \sum_{i=1}^{4} L_{B,i} + L + S_h \Rightarrow S_h \geq 3 \cdot 2.000 \text{ m} + 294 \text{ m} + 150 \text{ m} \iff S_h \geq 6.444 \text{ m}
\]

The necessary headway time \( t_h \) between two IC3 trains driving at a constant speed \( v = 50 \text{ m/s} \) (=180 km/h) can consequently be determined by means of formula 4.6:

\[
t_h = \frac{S_h}{v} = \frac{6.444 \text{ m}}{50 \text{ m/s}} = 129 \text{ s}
\]

You can use formula 4.1 to calculate the fundamental daily capacity \( K_f \), when it is operated in the above way:

\[
K_f = u \cdot \frac{\Delta T}{t_h} \Rightarrow K_f = 60\% \cdot \frac{1 \text{ h} \cdot 3.600 \text{ s/h}}{129 \text{ s/train}} = 16 \text{ trains/h}
\]

It should be noticed at the above calculations refer to a system, where the trains are driving at a constant speed (180 km/h) and the block section is \( L_B = 2.000 \text{ m} \). The areas around the station, where the trains are going to stop and therefore run at a lower speed, have not been taken into account. Nor do the calculations take into account the time.
for setting up the train route, the cancellation of the train route and the signal realization time, which means that the real headway time is approximately 10 – 15 seconds longer. However, the importance is marginal and will at the most mean 1 train/h less than calculated.

4.3 Wiggly wire

Figure 4.4 shows a situation where the Re-train control system is designed in such a way that information about the signalling on the next signal is given continuously in the driver’s cab. The Danish ATC system is designed as a discreet train control system, which means that the exchange of data information only takes place at the passage of balises. However, the ATC can be made continuous by providing the infrastructure with a wiggly wire whereby a continuous flow of information flow is obtained. For capacity reasons this will often be preferred on lines with dense traffic, as the continuous ATC system all things being equal gives rise to an increased capacity reserve as compared to the discreet ATC system.

![Figure 4.4 Train sequence at discreet block and continuous ATC (wiggly wire) [2].](image)

According to the same principle as in formula 4.4, the headway distance $S_h$ can be determined in the following way:

Formula 4.12 $S_h \geq S_b + B_i + L + S_s$

Provided $S_b = \sum_{i=2}^{n} B_i$ formula 4.4 and formula 4.12 become identical. In this case, it will have no effect to provide the ATC system with a wiggly

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wire, as the headway distance $S_h$ will be optimized with block lengths $L_b$ that have been adapted to the braking distance $S_b$

The optimal speed $v_{opt}$ can be determined by inserting formula 4.12 ($\geq$ is replaced by $=$) in formula 4.6, and then differentiate the expression with regard to the time:

\[
t_h = \frac{1}{2 \cdot (c \cdot r_h + r_g)} \cdot \frac{v^2 + 0.008s/m \cdot L + 3s \cdot v + L_{B,1} + L + S_s}{v} \]

\[
t_h' = \frac{1}{2 \cdot (c \cdot r_h + r_g)} \cdot \frac{L_{B,1} + L + S_s}{v^2} \]

If $t_h' = 0$ the optimum speed $v_{opt}$ is found:

\[
v_{opt}^2 = 2 \cdot (c \cdot r_h + r_g) \cdot (L_{B,1} + L + S_s) \]

\[
\Rightarrow \text{Formula 4.13 } v_{opt} = \sqrt{2 \cdot (c \cdot r_h + r_g) \cdot (B_l + L + S_s)}
\]

4.3.1 Example of calculation with wiggly wires

If the same basic data as in section 4.2.1 Example of calculation are used, however with wiggly wires on the entire line, the headway distance can be determined in the following way:

\[
S_h \geq S_h + L_{B,1} + L + S_s \Rightarrow S_h \geq 2.379m + 2.000m + 294m + 150m \Rightarrow S_h \geq 4.823m
\]

The minimum headway time $t_h$ can then be determined to:

\[
t_h = \frac{S_h}{v} = \frac{4.823m}{50m/s} = 97s
\]

Which gives the following fundamental daily capacity $K_t$:

\[
K_t = u \cdot \frac{\Delta T}{t_h} \Rightarrow K_t = 60\% \cdot \frac{1h \cdot 3.600s/h}{97s/train} = 22 \text{ trains/h}
\]

The effect of wiggly wires will to a large extent depend on how well the block sections have been adapted to the necessary braking distance. This issue is further discussed in section 4.5. In the present example of calculation the wiggly wires are very important, as the block sections
have been very poorly adapted as compared to the dimensioning braking distance. Without the wiggly wires the train thus needs 4.000 m (equivalent to 2 blocks) to brake, where it would have been sufficient with 2.379 m corresponding to the actual braking distance of the train.

The optimum speed $v_{opt}$ of the trains operating in the planned system can be calculated as:

$$v_{opt} = \sqrt{2 \cdot \left( \frac{61 \cdot 184 + 61}{1200} \right) \cdot \left( 9.82 \text{ m/s}^2 \cdot (-1\%) \right) \cdot \left( 2.000 \text{ m} + 294 \text{ m} + 150 \text{ m} \right)}$$

$$v_{opt} = 53.8 \text{ m/s} = 194 \text{ km/h}$$

As a speed of 180 km/h is assumed, it more or less corresponds to the optimum speed, as the headway time $t_h$ is very insensitive around its optimum. Thus, when inserting $v = 180$ km/h and $v_{opt} = 194$ km/h respectively, it is seen that the change in headway time is $\Delta t_h = 0.2$ s. On the other hand, if the speed is $v = 120$ km/h, the change in headway time will be $\Delta t_h = 10.6$ s if it is compared with the optimum speed $v_{opt} = 194$ km/h.

### 4.4 Moving block

Presently, efforts are being made at the European level to develop a system that can improve the capacity (i.e. reduce the necessary headway time) without reducing the safety or the speed of the train. The system in question is called ETCS\textsuperscript{10}, and when it has been fully developed it may appear as described in figure 4.6.

\textsuperscript{10} European Train Control System
The system has been designed so that the driver is informed about the permitted speed and signal messages via the ETCS system which is based on radio communication between the individual trains and some transmitting stations. ETCS will also work as a control system ensuring that speed restrictions are not exceeded. All external signals will thus be superfluous when an active ETCS system has fully been implemented. In this way, the headway distance $S_h$ can be reduced to:

Formula 4.14 $S_h = S_b + S_s + L$

If formula 4.12 is rewritten so that the term $L_{B,1}$ is eliminated, the expression will describe the optimum speed $v_{\text{opt}}$, when moving blocks are used.

Formula 4.15 $v_{\text{opt}} = \sqrt{2 \cdot (c \cdot r_b + r_g) \cdot (L + S_s)}$

If one could imagine a situation where relative braking distances were used in road traffic, the headway distance $S_h$ could be further reduced. A relative braking distance means that the danger point in front of train (the train in front) is also moving. Two trains going at the same speed could therefore in principle drive very closely, only separated by a safety distance $S_s$, under the assumption that the train in front will not momentarily reach the speed of 0, but has a normal slackening of speed. The described situation is shown in figure 4.6.
It appears from figure 4.6 that the headway time $S_h$ can be determined in the following way:

Formula 4.16 $S_h = S_s + L$  provided $S_{b,1} \geq S_{b,2}$

otherwise $S_h = S_{b,2} - S_{b,1} + S_s + L$

Where:  $S_{b,1}$ is the braking distance for train 1 [m]  
        $S_{b,2}$ is the braking distance for train 2 [m]

Therefore, the headway distance $S_h$ will almost only depend on the length $L$ of the train in front!!

However, this system will be quite a revolution in the railway world, as the railway is based on very big safety requirements. So the horizon for the introduction of such a system is probably very remote.

However, it would be very interesting to introduce this system, e.g. in case of reduced speed in connection with addition and/or declutching of train units. In this way it will be possible to split IC3 trains with different destinations for the individual train units, while it is driving. Apart from increasing the capacity, it will make it possible to reduce
Determination of headway times

the travel time by approximately 5 minutes per addition or declutching of train units.

4.5 Example of calculation with moving block

If the example of calculation shown in section 4.2.1 and section 4.3.1 is carried out with a moving block, the following necessary headway distance $S_h$ is obtained:

\[ S_h \geq S_b + S_s \Rightarrow S_t \geq 2.379m + 294m + 150m \Leftrightarrow S_h \geq 2.823m \]

The necessary headway time $t_h$ thus becomes:

\[ t_h = \frac{S_h}{v} = \frac{2.823m}{50m/s} = 57s \]

This gives the following fundamental daily capacity $K_f$:

\[ K_f = u \cdot \frac{\Delta T}{t_h} \Rightarrow K_f = 60\% \cdot \frac{1h \cdot 3.600s/h}{57s/train} = 38 \text{trains/h} \]

It is no surprise that the example of calculation shows that moving blocks can increase the capacity considerably. This is due to the fact that the “superfluous” block, which with discreet blocks ensures that the train does not have to start its gradual braking unnecessarily, does not exist in case of moving blocks. With moving blocks it will primarily be the speed of the trains (or more correctly the braking distance) that determines the necessary headway distance $S_h$. Therefore, it will be very suitable to introduce moving blocks when the speed is low.

This can further be illustrated by determining the optimum speed $v_{opt}$:

\[ v_{opt} = \sqrt{2 \cdot \left(70\% \cdot \frac{6.1 \cdot 184 + 61}{1200} \text{m/s}^2 + 9.82 \text{m/s}^2 \cdot (-1\%)) \cdot (294 \text{ m} + 150 \text{ m}) \right)} \]

\[ v_{opt} = 22.9 \text{ m/s} = 83 \text{ km/h} \]

It appears from the above that it is particularly in case of local traffic/urban railways, where the speed is normally below 100 km/h,
that the principle of moving blocks can be used with maximum effect. For urban railways the length of the train $L$ will normally not exceed 100 m, and the safety distance $S_s$ will also be reduced to about 80 m. This means that the optimum speed will be about 60 km/h.

### 4.6 Optimizing of the block length(s) on a line

As briefly indicated in 4.3 it is to some extent possible to minimize the headway time (and thereby increase the line capacity), if the block distances are adapted to the braking distances of the trains. This however requires that the level of service on the line in question is more or less homogeneous. This means that the trains operating the actual line should more or less have the same braking properties and stopping patterns. If this is not the case, it is necessary to focus on the most frequent ways of operation, but in this way there is a risk that the block section will be very unsuitable for a small number of trains. If the level of service is very inhomogeneous, it is better to choose an ordinary standard block length which is primarily based on the maximum permitted speed on the line.

Figure 4.7 thus shows how the permitted speed on the line influences the required headway time $t_h$:

![Speed vs Headway Time Graph](image)

**Figure 4.7** The headway time $t_h$ as function of the trains’ speed $v$.

The indicated headway time $t_h$ as function of the trains’ speed $v$ has been determined based on a situation, where the block section is carefully adapted to the driving behaviours of a long IC3 train ($L = 320$ m) at different speeds on a horizontal line. The infrastructure is equipped with external signals and discreet ATC (i.e. no wiggly wire)
Determination of headway times

as shown in figure 4.3. However, as the braking distance $S_b$ has been carefully adapted to the block lengths, it is equivalent to the system being fitted with wiggly wire, since $L_{B,2}$ (and potentially $L_{B,3}, L_{B,4} \ldots \ldots L_{B,n}$) can be replaced by the braking distance $S_b$ in formula 4.4.

As appears from figure 4.7, the smallest theoretical headway times can be obtained if the trains are operated at a speed of 60 – 70 km/h and the block section is adapted accordingly. In this way it is possible to obtain headway times of about 60 s, if driving on 3 green lamps (i.e. the braking distance $S_b$ is equivalent to the length of 2 block sections), whereas the theoretical necessary headway time $t_h$ can be as low as 70s, if the infrastructure is designed in such a way that the braking distance $S_b$ is equivalent to one block length, and driving on “2 green”. By letting the braking distance correspond to the sum of many block sections you get in principle a system that is equivalent to system with a moving block cf. figure 4.6. However, in practice it will be difficult to design a system with more than “3 green”, if external signalling is used.

If you want to determine which block section is the dimensioning one on a line, you can calculate the block occupation time $t_B$ for each individual section. If the train stops at stations that delimit the studied open line, the speed $v$ will not be the same in all the block sections. Provided the same block length has been chosen on the entire line, the first and/or last block section will be dimensioning for the headway time.

Studies [1] show that when a permanent block length of 2.500 m is replaced by gradual “upgrading and downgrading” of the block lengths in connection with entry and exit from the stations, the theoretically smallest headway distance $t_h$ is reduced considerably.

Figure 4.8 shows an example where a 25 km long line section has been divided with permanent block distance and adapted block distance, respectively (0,5 km; 1,0 km; 1,5 km; 2,0 km; 2,5 km.......2,5 km; 2,0 km; 1,5 km; 1,0 km; 0,5 km)

<table>
<thead>
<tr>
<th>Regular length of block sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{B,1}$ $L_{B,2}$ $L_{B,3}$ $L_{B,4}$ $L_{B,5}$ $L_{B,6}$ $L_{B,7}$ $L_{B,8}$ $L_{B,9}$ $L_{B,10}$</td>
</tr>
<tr>
<td>Station     Station</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Adjusted block sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_{B,1}$ $L_{B,2}$ $L_{B,3}$ $L_{B,4}$ $L_{B,5}$ $L_{B,6}$ $L_{B,7}$ $L_{B,8}$ $L_{B,9}$ $L_{B,10}$ $L_{B,11}$ $L_{B,12}$ $L_{B,13}$ $L_{B,14}$</td>
</tr>
<tr>
<td>Station     Station</td>
</tr>
</tbody>
</table>

Figure 4.8 Permanent contra adapted block section.
With the above adjustments it is possible to obtain a reduction of the dimensioning headway time $t_h$ from just under 3 minutes with the permanent block section to just under 2 minutes in case of the adapted block section, provided the line is operated with IC3 trains driving at a maximum speed of approx. 180 km/h [1]. The dimensioning block sections are the black ones. All other things being equal, it will therefore increase the capacity to use short block sections in connection with entry and exit from stations where halts can take place. However, the block lengths must be adapted to the dimensioning braking distances, otherwise the reduction of the block length may hinder the capacity!!!(cf. figure 4.9).

If the example of calculation from section 4.2.1 Example of calculation is carried out with block lengths $L_B = 2.500 \text{ m}$, it is seen that it gives rise to a practical capacity increase $\Delta K_f = 3\text{ trains/h}$, when driving on 2 "green" and not 3 "green", which is the case with block lengths $L_B = 2.000 \text{ m}$, as the braking distance is $S_b = 2.379 \text{ m}$.

In the above cases, block lengths of $L_B = 2.500 \text{ m}$ would be more favourable than block lengths of $L_B = 2.000 \text{ m}$. This is due to the fact that short blocks make it necessary to "reserve" 4.000 m for the braking distance, whereas 2.500 m is sufficient for long blocks.

The above phenomenon is reproduced in figure 4.9 that clearly shows that a reduction of the block length can result in a considerable increase of the headway time, provided the reduction means that an extra block section is to be reserved for the braking distance. The figure next to the graph show how many block sections that must be reserved for the braking distance.

It appears that the increase in capacity is biggest in the area between 1 and 2 block sections for the braking distance, whereas the increase in capacity is hardly noticeable if the braking distance covers several block sections (i.e. close to the concept of moving block).
Determination of headway times

Minimum headway time

Length of block sections

Figure 4.9 The headway time as function of block length.

Therefore, when dimensioning the block lengths you have to be very careful not to reduce the block length to the absolute minimum. A consequence of this could be that a small future increase of the line speed could totally ruin the optimizing, as it will be necessary to use an extra block section for the gradual braking, as compared to the original plan.

References
[3] Rail Net Denmark, SODB – Interlocking systems and how they are operated, Continuously updating

Determination of headway times
Chapter 5: Capacity on double track lines

In the previous chapter (Determination of headway times) calculation rules in systems with homogeneous traffic were established. Such ideal situations rarely occur, why it will normally be interesting to observe, how the traffic can be carried out if it is not homogeneous. Therefore, the focus in this chapter will primarily be on cases with mixed traffic, where the double track line is operated by both fast and slow trains.

The operation patterns of double track lines will normally be such that each track is reserved for one direction of travel. In Denmark, you will normally use the right track\(^1\) as known from road traffic, but driving on the left track can occur. However, this will normally only be relevant in case of operation irregularities, e.g. due to line obstructions. If the left track is used, it will generally imply a reduced maximum speed, as the design of the safety system for the secondary direction of travel is not quite as sophisticated. Equally the block sections will often be very long, which also reduces the capacity.

Recently the use of single-line block system with directional control\(^2\) has however been introduced on certain lines with a high capacity utilization. On these lines it is however preferred to timetable the traffic on the right track, so that there do not arise unnecessary conflicts in connection with the shift between right-hand and left-hand line running. Therefore, the single-line block system with directional control does not do justice to itself until it is used in connection with line obstructions and the like, where the traffic can then be operated as on a technically well-equipped single track line.

When carrying out capacity studies, it is in case of a long double track line expedient to split the line into several small line segments where the capacity calculations can be done “manually”. In figure 5.1 is shown a line with a junction station (8), where it would be obvious to to split the line. In this way you get 3 line segments, each of which can be examined. Furthermore, it will be suitable to examine the junction station separately, as there will be crossing streams of traffic on this station that may reduce the capacity.

---

1 In Sweden however, driving on the left is the rule.

2 With a single-line block system with directional control, it is possible to use both left-hand and right-hand line running on all the tracks.
Apart from the above splitting and in case you want to make a capacity analysis, it will therefore be suitable to split the line segments, so that there will be no overtaking in the analysis area. In case of planned operations, the cross-overs between the main lines will only be used in very rare occasions, and the analysis areas for one of the directions of travel will therefore be station 1 → station 3, station 3 → station 5 etc., whereas it for the other direction of travel will be station 8 → station 6, station 6 → station 4 etc. Afterwards, it is possible to identify the bottlenecks in each of the splitted areas. A bottleneck will result in a very high headway time $t_n$.

If the system contains a spot where the bottlenecks are much bigger than at the other critical spots, this bottleneck will be determining for the operation. Since the trains in case of maximum utilization will practically stand in a line to pass this bottleneck, the practical capacity $K_f$ will almost be equal to the theoretical capacity $K_{\text{max}}$. This however requires that the trains are not brought to stop just before the bottlenecks.

### 5.1 Inhomogeneous traffic

If the average speed of the individual trains on a double track line is very different, it is necessary to carry out overtaking if you want to obtain optimum capacity utilization.

Figure 5.2 shows in a graphic timetable how a fast train will catch up on a slow one.
Figure 5.2 Graphic timetable with maximum mixed traffic at 2 velocity profiles.

On the illustrated railway line, the 3 fast and the 3 slow trains will therefore occupy the period $t_{\Sigma,{\text{mix}}}$. It appears from figure 5.2 that the block sections at the 2 stations will be decisive for how small a headway that can be obtained. These block sections can thus with advantage be adapted to the traffic driving on the line as shown in figure 5.3.

Figure 5.3 Optimum block sections at the headway time fast IC train followed by slow Re-train.
Capacity on double track lines

Above is shown a situation, where a slow Re-train is about to leave a station just after overtaking by a fast IC-train. The conditions are optimized as there is no “buffer” between the so-called Sperrtreppen\(^3\) for the 2 trains. If block \(L_{B,1}\) is prolonged, this block section alone will be dimensioning for the headway time, whereas the station block \(L_{B,0}\) and block \(L_{B,2}\) will be the dimensioning one, if block \(L_{B,1}\) is reduced.

The speed \(v_{IC}\) of the fast IC train can be expressed in the following way:

Formula 5.1  \[ v_{IC} = \frac{L_{B,i}}{t_{k,i,IC}} \]

Where: \(B_i\) is the length of the \(i^{th}\) block section  
\(t_{k,i,IC}\) is the IC-train’s running time through the \(i^{th}\) block section

In the same way the speed of the slow Re-trains \(v_{Re,k}\) can be calculated, when it drives at a constant speed.

Formula 5.2  \[ v_{Re,k} = \frac{B_i}{t_{k,i,Re}} \] for \(1 < i \leq n\)  
(it is a prerequisite that the Re-train has obtained \(v_{Re,k}\) after having passed the first block section and maintain this through all \(n\) block sections)

Where: \(t_{k,i,Re}\) is the running time of the Re-train through the \(i^{th}\) block section

The following will apply to the headway time \(t_{h,i}\) between a fast IC train and a slow Re-trains at the entry into the \(i^{th}\) block section:

Formula 5.3  \[ t_{h,i} < t_{h,i+1} \] (for fast train followed by slow train)

To be able to determine the dimensioned headway time in case of inhomogeneous traffic, the conditions around the entry to an overtaking station, where the slow Re-trains can be overtaken by the fast IC trains, must also be taken into consideration.

\(^3\)The “Sperrtreppen” (The German term is “Sperrtreppen” which is used as the official word in several languages, including Danish) symbolize the physical occupation of the block.
Figure 5.4 Optimum block sections at the headway time slow train followed by fast train.

Formula 5.1 and formula 5.2 will also apply in connection with entry to the overtaking station. As appears from figure 5.4, the headway time has however now changed to a slow Re-train followed by a fast IC-train, and therefore the following will apply to the headway time \( t_{h,i} \) at the entry into i'th block section:

\[
\text{Formula 5.4 } t_{h,i} > t_{h,i+1} \quad \text{(for slow train followed by fast train)}
\]

If we look at figure 5.2 it appears that period \( t_{\Sigma,mix} \), which is required to operate \( N \) cycles of a maximum mixed traffic (at 2 different velocity profiles), can be found in the following way:

\[
t_{\Sigma,mix} = N \left( t_{k,0,Re} + t_{k,i,Re} - \frac{L_{B,1}}{v_{IC}} + \sum_{i=2}^{n-1} L_{B,i} \left( \frac{1}{v_{Re,k}} - \frac{1}{v_{IC}} \right) + t_{k,n,Re} + t_{k,n+1,Re} \right)
\]

\[
\%
\]

\[
\text{Formula 5.5 } t_{\Sigma,mix} = N \left( t_{k,0,Re} + \sum_{i=1}^{n-1} \left( t_{k,i,Re} - t_{k,i,IC} \right) + t_{k,n,Re} + t_{k,n+1,Re} \right)
\]

(It is is assumed that the braking distance \( S_{b,IC} \) of the IC-train is not longer than \( L_{B,n} \))
Capacity on double track lines

Where: $t_{k,n+1,\text{Re}}$ is the time it takes the Re-train to go from the entrance signal until it has passed the fouling point\textsuperscript{4} with its entire length $L$

In order to minimize $t_{\text{c,mix}}$ you primarily have to:

- Minimize the distance between overtaking stations (i.e. minimize $\Sigma L_B$)
- Reduce the speed difference of the 2 different velocity profiles
- Create good entry and exit conditions at the overtaking stations (i.e. maximize $v$)

Figure 5.5 thus shows that the headway time can be reduced $\Delta t_h$ by creating good entry conditions on an overtaking station.

The unbroken line (in figure 5.5) shows how the train that is about to be overtaken can continue at unaltered speed through the switch taking the deflecting section. Therefore, the braking is not started until it is necessary due to the halt. On the other hand, the broken line shows that the speed must normally be reduced before a switch for trains driving in the deflecting section. By using very slim switches, it is possible to obtain high speeds on the deflecting section of the switch.

\textsuperscript{4} In this case, the fouling point which releases the main train route at the entrance of the station.
For instance a switch with a crossing relation 1:26.5 allows a speed of 130 km/h through the deflecting section of the switch.

By sorting the mixed traffic into various pools, where Re-trains with the same average speed run just after each other, a better capacity utilization can be obtained. When comparing figure 5.2 and figure 5.6, it appears very clearly that the operation of several trains can be limited if it is possible to bundle the trains.

Figure 5.6 Graphic timetable with mixed traffic arranged in pools.

The period $t_{\Sigma,\text{pool}}$ that is required to perform an operation situation with pools, can be determined in the following way:

$$t_{\Sigma,\text{pool}} = Z \left( t_{h,\text{IC}} + \sum_{i=1}^{n-1} (t_{k,i,\text{IC}} - t_{k,i+1,\text{IC}}) + t_{k,n,\text{IC}} + t_{k,n+1,\text{IC}} + (N_{\text{IC}} - 1)t_{h,\text{IC}} + (N_{\text{Re}} - 1)t_{h,\text{Re}} \right)$$

Where:
- $N_{\text{IC}}$ is the number of IC trains in a pool
- $N_{\text{Re}}$ is the number of Re-trains in a pool
- $Z$ is the number of cycles consisting of a pool of IC trains and a pool of Re-trains
- $t_{h,\text{IC}}$ is the minimum headway time between 2 IC trains on the line
- $t_{h,\text{Re}}$ is the minimum headway time between 2 Re-trains on the line

In order to examine and compare how well the traffic is arranged in pools with similar average speed, you can introduce a bundling coefficient $\rho$, in the following way:

$$\rho = \frac{n_{\text{Re}}/n_{\text{IC}}^2 - n_{\text{Re}}/n_{\text{IC}}}{n_{\text{Re}}/n_{\text{IC}}}$$
Capacity on double track lines

Where:  
- $n_{Re/Re}$ is the number of headways, when 2 Re-trains run just after each other
- $n_{IC/IC}$ is the number of headways, when 2 IC trains run just after each other
- $n_{Re/IC}$ is the number of headways with the order Re followed by IC (or reversed)
- $n_{Re}$ is the number of Re-trains
- $n_{IC}$ is the number of long-distance motor-coach train

In case of the weakest "bundling" (as shown in figure 5.2), the bundling degree $\rho_{\text{min}} = -1$, as $n_{Re/Re} = n_{IC/IC} = 0$ and $n_{Re/IC} \cdot n_{Re/IC} = n_{Re} \cdot n_{IC}$

The strongest bundling will approach the bundling degree $\rho_{\text{max}} = 1$, but it will be practically impossible to obtain the value 1, as the maximum bundling degree $\rho_{\text{max}}$ can be revalued in the following way:

$$\rho_{\text{max}} = \frac{(n_{Re} - 1) \cdot (n_{IC} - 1) - 1}{n_{Re} \cdot n_{IC} - n_{Re} \cdot n_{IC}} = 1 - \frac{1}{n_{Re}} - \frac{1}{n_{IC}}$$

For very big values of $n_{Re}$ and $n_{IC}$, the bundling degree $\rho_{\text{max}}$ will thus be very close to 1.

The totally arbitrary bundling is characterised by giving the bundling degree $\rho_{\text{arb}} = 0$, which means that in these situations the following will apply:

$$\rho_{\text{arb}} = \frac{n_{Re/Re} \cdot n_{IC/IC} - n_{Re/IC}^2}{n_{Re} \cdot n_{IC}} = 0$$

\[\uparrow\]

Formula 5.9  
$n_{Re/Re} \cdot n_{IC/IC} = n_{Re/IC}^2$ (in case of perfect bundling, i.e. $\rho_{\text{arb}} = 0$)

If you want to make a capacity analysis, it will, in case of more than 2 train categories, be expedient to write down in a table all the headway combinations that may occur as shown table 5.1. The table includes the dimensioning headway time $t_{h,x,y}$ at all combinations of train category $x$ followed by train category $y$. It must always be indicated with starting point in the same reference point, e.g. the entrance station with regard to that the train can be operated to the first
overtaking and/or junction station without any conflicts. The number \( n_{x,y} \) of the individual combinations occurring during the period \( T \) is also to be indicated. The size of these numbers will of course depend on the actual timetable, where the mutual order of the trains is known.

The individual headway times \( t_{h,x,y} \) can be found by a generalization of formula 5.5, so that e.g. Re is replaced by \( x \) and IC by \( y \). It is however important to underline that you can of course not get negative headway times. If the entrance station is used as reference, the dimensioning headway times \( t_{h,x,y} \) could be determined based on the following knowledge:

\( v_x > v_y \): The dimensioning headway time can be found in the first block section (the station block at the entrance station)

\( v_x = v_y \): The dimensioning headway time will normally be found in the longest block section

\( v_x < v_y \): The dimensioning headway time is the time difference of the trains between the entrance station and the first overtaking possibility. To this should be added the time it takes the train \( x \) to clear the last block section (the station block at the overtaking station).

### Table 5.1 Example of headway schedule [2].

<table>
<thead>
<tr>
<th>Following train ( y ) →</th>
<th>Train category 1 (e.g. high-speed train)</th>
<th>Train category 2 (e.g. IC-train)</th>
<th>Train category 3 (e.g. Re-train)</th>
<th>Train category 4 (e.g. freight train)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trains in front ( x ) ↓</td>
<td>( t_{h,1,1} = 2\frac{1}{2} \text{ min} ) ( n_{1,1} = 0 )</td>
<td>( t_{h,1,2} = 2\frac{1}{2} \text{ min} ) ( n_{1,2} = 2 )</td>
<td>( t_{h,1,3} = 2\frac{1}{2} \text{ min} ) ( n_{1,3} = 0 )</td>
<td>( t_{h,1,4} = 2\frac{1}{2} \text{ min} ) ( n_{1,4} = 0 )</td>
</tr>
<tr>
<td>Train category 2 (e.g. IC-train)</td>
<td>( t_{h,2,1} = 3\frac{1}{2} \text{ min} ) ( n_{2,1} = 1 )</td>
<td>( t_{h,2,2} = 2\frac{1}{2} \text{ min} ) ( n_{2,2} = 0 )</td>
<td>( t_{h,2,3} = 2\frac{1}{2} \text{ min} ) ( n_{2,3} = 1 )</td>
<td>( t_{h,2,4} = 2\frac{1}{2} \text{ min} ) ( n_{2,4} = 1 )</td>
</tr>
<tr>
<td>Train category 3 (e.g. Re-train)</td>
<td>( t_{h,3,1} = 7 \text{ min} ) ( n_{3,1} = 0 )</td>
<td>( t_{h,3,2} = 6 \text{ min} ) ( n_{3,2} = 2 )</td>
<td>( t_{h,3,3} = 3 \text{ min} ) ( n_{3,3} = 2 )</td>
<td>( t_{h,3,4} = 3 \text{ min} ) ( n_{3,4} = 1 )</td>
</tr>
<tr>
<td>Train category 4 (e.g. freight train)</td>
<td>( t_{h,4,1} = 9 \text{ min} ) ( n_{4,1} = 0 )</td>
<td>( t_{h,4,2} = 8 \text{ min} ) ( n_{4,2} = 0 )</td>
<td>( t_{h,4,3} = 5 \text{ min} ) ( n_{4,3} = 2 )</td>
<td>( t_{h,4,4} = 3\frac{1}{2} \text{ min} ) ( n_{4,4} = 0 )</td>
</tr>
</tbody>
</table>
Capacity on double track lines

The average headway time $t_{h,a}$ can then be determined based on data equivalent to those in table 5.1:

$$t_{h,a} = \frac{\sum_{(x,y)=(1,1)}^{(k,k)} n_{x,y} t_{h,x,y}}{\sum_{(x,y)=(1,1)}^{(k,k)} n_{x,y}}$$

Formula 5.10

Where: $k$ is number of train categories

### 5.1.1 Example of calculation with mixed traffic

A 20 km double track line delimited by 2 overtaking stations is operated by throughgoing IC-trains driving at a speed $v_{IC} = 180$ km/h and stopping regional trains driving at a maximum speed $v_{Re,k} = 140$ km/h. A MZ-regional train needs approx. 7.3 km to accelerate from 0 to 140 km/h. With a station block $L_{B,0} = 800$ m, the regional train set will reach maximum speed after 6.5 km on the line. The entire acceleration will take 258 seconds. The last block length $L_{B,n} = 2.000$ m is assumed to be long enough for the IC-train to brake within this block. A MZ-regional train has a braking distance of approx. 1.200 m from 140 km/h to 0. It is assumed that this braking distance can start at the entrance signal and be finished at the platform when the entire train has passed the fouling point.

With a timetable equivalent to the one shown in figure 5.2, where 3 regional trains and 3 IC trains operate the line with alternately fast and slow trains, the necessary period $t_{\Sigma, mix}$ to operate this timetable can be determined by means of formula 5.5 as follows:

$$t_{\Sigma, mix} = 3 \cdot \left[ 258 \text{s} \frac{6.500\text{ m}}{180\text{ km/h}} + 11.000\text{ m} \left( \frac{1}{140\text{ km/h}} - \frac{1}{180\text{ km/h}} \right) + \frac{2.500\text{ m}}{140\text{ km/h}} + 60\text{s} \right]$$

\[ t_{\Sigma, mix} = 945\text{s} \approx 15.8 \text{ min} \]

It should be noticed that above calculation is based on ideal conditions (cf. figure 5.3 and figure 5.4), where the trains around entry or exit to and from the overtaking station can be “packed” as closely as theoretically possible. Furthermore, it is assumed that there are no capacity limitations on the adjacent lines.
If the above plan of operation is carried out based on the principle of pools (cf. figure 5.6), the necessary period \( t_{\Sigma, \text{pool}} \) to operate this timetable according to formula 5.6 will be:

\[
t_{\Sigma, \text{pool}} = 258s - \frac{6.500m}{180km/h} + 11.000m \cdot \left( \frac{1}{140km/h} - \frac{1}{180km/h} \right) + \frac{2.500m}{140km/h} + 60s + (3-1) \cdot 90s + (3-1) \cdot 90s
\]

\[
= \frac{600m}{180km/h} + 11.000m \cdot \left( \frac{1}{140km/h} - \frac{1}{180km/h} \right) + \frac{2.500m}{140km/h} + 60s + 90s + 90s
\]

\[
= 675s \approx 11.3 \text{min}
\]

The calculation is based on headway times between similar train classes of 90 seconds which is in good compliance with the results from section 4.3.1. The above calculations show that the capacity need can be reduced by approx. 30\% provided the timetable is compressed with pools instead of a timetable with maximum mixed traffic.

By means of formula 5.7, the bundling degree \( \rho \) for each of the 2 cases can be calculated in the following way:

**Bundling degree in figure 5.2:**

\[
\rho_{\text{mix}} = \frac{-3^2}{3 \cdot 3} = -1
\]

**Bundling degree in figure 5.6:**

\[
\rho_{\text{pool}} = \frac{2 \cdot 2 - 1^2}{3 \cdot 3} = 1/3
\]

Thus, in the timetable in figure 5.2 the bundling degree is minimal, whereas the timetable in figure 5.6 is bundled randomly, as the bundling degree is positive.

In most cases there will be more than 2 velocity profiles for the train categories driving on a line. In these situations the timetable offers a large number of combination possibilities as to how the timetable can be composed with regard to the mutual order of the trains. In this case, a table like table 5.1 is therefore suitable to indicate the individual headway times and their occurrence. Table 5.1 thus describes a timetable with the following headway:

High-speed \( \rightarrow \) IC \( \rightarrow \) Re \( \rightarrow \) Re \( \rightarrow \) Freight \( \rightarrow \) Re \( \rightarrow \) IC \( \rightarrow \) High-speed \( \rightarrow \) IC \( \rightarrow \) Freight \( \rightarrow \) Re \( \rightarrow \) Re \( \rightarrow \) IC

If the values from table 5.1 are inserted in formula 5.7, the average headway time \( t_{h,a} \) for above timetable is calculated to:
Capacity on double track lines

\[ t_{h,a} = \frac{2 \cdot 2\frac{1}{2} \text{min} + 3\frac{1}{2} \text{min} + 2\frac{1}{2} \text{min} + 2\frac{1}{2} \text{min} + 2 \cdot 6 \text{min} + 2 \cdot 3 \text{min} + 3 \text{min} + 2 \cdot 5 \text{min}}{12} \]

\[ t_{h,a} = 3.7 \text{min} \]

It is convenient to use the average headway time \( t_{h,a} \) when comparing 2 timetables containing the same trains, but which have been combined in different ways. If the 2 examples are compared with maximum mixed traffic and pools, respectively, the practical capacity \( K_f \) during a peak hour can be found in the following way:

**Maximum mixed traffic:**

\[ K_f = 0.75 \frac{60 \text{min}}{15.8 \text{min}/6 \text{trains}} = 17 \text{trains} \]

**Pools:**

\[ K_f = 0.75 \frac{60 \text{min}}{11.3 \text{min}/6 \text{trains}} = 24 \text{trains} \]

Experience from Danish double track lines with mixed traffic shows however that if you want the timetable to operated with a fair quality\(^5\), the maximum number of trains that can be operated during the peak hour are 17 trains/h in each direction. However, to a large extent the practical capacity \( K_f \) will depend on the construction of the infrastructure and on the homogeneity of the traffic. The above example of calculation is based on very idealized conditions. For instance it does not include the regional trains’ stops at stations/halts where overtaking is not possible.

### 5.2 Stations

Often the stations require a separate capacity study, as the specific conditions of these stations can determine how the traffic can be operated. Especially at stations, where the stopping pattern varies depending on e.g. the train class, it will be interesting to examine whether overtaking is possible and whether it influences the traffic operation. On certain stations it will also be necessary to examine whether crossing or coincident traffic flows will lead to capacity problems that can be dimensioning for the operation.

#### 5.2.1 Halts

When a train makes a halt, the total travel time will of course be extended, which can be described in the following way:

---

\(^5\) The concept of “fair quality” must of course be defined by means of measurable elements which is done in chapter 7 (timetabling)
Formula 5.11  \( \Delta t_{st} = t_{dec} + t_{dwell} + t_{acc} - t_0 \)

Where:  
\( \Delta t_{st} \) is the travel time extension due to the halt  
\( t_{dec} \) is the time that the train decelerates  
\( t_{dwell} \) is the time that the train stands still  
\( t_{acc} \) is the time that the train accelerates  
\( t_0 \) is the time it takes a throughgoing train to cover the line where the speed must be reduced because of a halt

Furthermore, in formula 5.11 you can add a positive term which is the running time, where the train drives at a constant speed when the deceleration has been started because of the halt. As shown in figure 5.5 it can be necessary to reduce the speed before a switch, whereby the train has to travel a distance at constant speed, or with reduced gradual braking. The same can occur in connection with the exit from the station.

Figure 5.7 Time elements during the stopping process.

Above is shown an optimum overtaking situation, where the stopping train shall not initiate the braking until it is necessary because of the halt. Furthermore, throughgoing trains can overtake the stopping train without decelerating. The deceleration time \( t_{dec} \) and the acceleration time \( t_{acc} \) will solely depend on the train’s driving behaviour provided the infrastructure does not give rise to any limitations in the form of speed reductions. On the other hand, the dwell time \( t_{st} \) will depend on
Capacity on double track lines

several factors. The main objective of the halt will normally be to exchange passengers or goods.

For freight train, the shunting of wagons and despatch service will normally take place at marshalling yards or freight stations, so that the traffic on the main lines is not disturbed, as the dwell times often last for several hours. Furthermore, due to reasons related to the construction of the timetable it will often be necessary to inflict some halts on the freight train with the only purpose of creating a generally optimum traffic operation, as the average speed of the freight trains will often be lower than that of many of the passenger trains.

For passenger trains, the dwell time will primarily be determined based on knowledge about how fast the passengers can be exchanged at the stations. In this case, the number of passengers plays an important role, but the design of the rolling stock and the technical equipment is certainly also important [1].

In situations with many passengers entering and/or leaving the train, the width of the doors and the access to these can extend the dwell time “unnecessarily”, provided they are not designed properly. For instance, the Danish IR4-trains (litra ER) have 2 double (= broad) doors placed next to each other, so that the outer doors do not become a bottleneck. On the other hand, all passengers have to pass a narrow passage to get from the outer doors to the seats, which can give rise to queues, if many passengers are to use the outer doors. However, in the IC3-trains these broad outer doors are placed separately meaning that the access to each door will take place from both sides thereby reducing the number of bottlenecks between the seats and outer doors. Furthermore, the separate location of the doors in the IC3 trains means that the average distance from a seat to the outer doors is reduced as compared to the IR4-trains, as the number of doors per train length is practically the same for the 2 types of trains [1].

The door closing and departure signal procedure also varies according to the rolling stock type. On local railways (e.g. S-trains), the departure procedure is normally very quick and only takes a few seconds, as the dwell time at the stations will normally determine the minimum headway time you can maintain. In Denmark, the length of the departure procedure varies considerably for white (IR4 trains and IC3-trains) and red rolling stock (e.g. Bn-wagons), respectively. In the red rolling stock the doors close as soon as the driver has ensured that the doors are free and the trains ready to depart. In the white rolling stock there is an acoustic signal at the doors after which the doors close.

At first sight, it may seem that the location of the doors and the technical flexibility can only reduce the dwell time by a few seconds, but at stations with many travellers it is reasonable to presume that
the dwell time can be reduced by at least ½ min, if the stock is designed optimally [1]. Consequently, on a line with 10 halts, travel time can be reduced by approx. 5 minutes. If this is opposed to the planned relocations of the lines, the arguments of which are based on travel size savings of the same order, it is quite interesting to focus on the entry and exit conditions etc. when you introduce rolling stock on lines, where you wish to obtain a travel time reduction.

Furthermore, the dwell times at stations can be established based on reasons related to the timetable construction, where a station halt can be extended to allow for connection to other trains (cf. chapter 7) or overtaking/crossing. Without the influence of such “special” conditions, the planned dwell time is normally set at 1 min., however ½ min. for stations with few travellers (e.g. Borup, Viby and Glumsø), whereas it on the biggest stations (e.g. the central stations) will normally be in the order of 2 – 3 min. (or even more) At stations where trains reverse direction or at the entrance and end stations, the dwell time will typically be in the order of 5 min., as time is needed to change the composition of the train, and the sales staff may need time to get new products on board.

5.2.2 Conflict points

At stations, where there will often be main routes that conflict with each other, it will in connection with a capacity analysis be relevant to examine whether this junction will give rise to a dimensioning bottleneck.

In figure 5.1, station 8 represents a station, where the traffic on the main routes cannot operate unimpededly, as 2 traffic flows gather/separate/cross at this junction. Depending on the design of the station, it will cause problems to a certain extent.

![Figure 5.8 Train route conflicts at junction station with right-hand line running.](image)

In figure 5.8 are shown 8 possible routes (a - h) for driving through the shown station. Each of these means that certain simultaneous
Capacity on double track lines

events cannot take place as part of the track is occupied. Table 5.2 thus shows which routes cannot be set at the same time:

<table>
<thead>
<tr>
<th>Routes</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>e</th>
<th>f</th>
<th>g</th>
<th>h</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>O</td>
<td>I</td>
<td>U</td>
<td>I/U</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>b</td>
<td>I</td>
<td>O</td>
<td>I/U</td>
<td>U</td>
<td>X₂</td>
<td>-</td>
<td>X₂</td>
<td>-</td>
</tr>
<tr>
<td>c</td>
<td>U</td>
<td>I/U</td>
<td>O</td>
<td>I</td>
<td>X₁</td>
<td>X₁</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>d</td>
<td>I/U</td>
<td>U</td>
<td>I</td>
<td>O</td>
<td>X₁/X₂</td>
<td>X₁</td>
<td>X₂</td>
<td>-</td>
</tr>
<tr>
<td>e</td>
<td>-</td>
<td>X₂</td>
<td>X₁</td>
<td>X₁/X₂</td>
<td>O</td>
<td>U</td>
<td>I</td>
<td>I/U</td>
</tr>
<tr>
<td>f</td>
<td>-</td>
<td>-</td>
<td>X₁</td>
<td>X₁</td>
<td>U</td>
<td>O</td>
<td>I/U</td>
<td>I</td>
</tr>
<tr>
<td>g</td>
<td>-</td>
<td>X₂</td>
<td>-</td>
<td>-</td>
<td>I</td>
<td>I/U</td>
<td>O</td>
<td>U</td>
</tr>
<tr>
<td>h</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>I/U</td>
<td>I</td>
<td>U</td>
<td>O</td>
</tr>
</tbody>
</table>

Table 5.2 Types of train route conflicts at junction station.

The letters in table 5.2 explain why the routes cannot be set at the same time, where the following abbreviations are used:

**O**: Overlapping routes

**I**: Converging of the 2 routes

**U**: Diverging of the 2 routes (normally not a capacity problem)

**X₁, X₂**: Crossing of the 2 routes at conflict point 1 or conflict point 2, respectively

**-**: No conflict between the 2 routes

The shown station thus contains $n_k = 46$ combination possibilities, where 2 routes of a total $n_Σ = 64$ possibilities cannot be set at the same time. If the station is designed with crossings out of grade at conflict 1 and/or conflict 2, the station will be less complex $ϕ$, as appears from table 5.3:
Crossings out of grade in figure 5.8

<table>
<thead>
<tr>
<th>Conflicting Combinations $n_k$</th>
<th>The station’s Complexity $\phi = n_k / n_\Sigma$</th>
<th>Conflict rate $\phi_0/\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>46</td>
<td>0,719 = $\phi_0$</td>
</tr>
<tr>
<td>“Conflict point” 1</td>
<td>38</td>
<td>0,594</td>
</tr>
<tr>
<td>“Conflict point” 2</td>
<td>38</td>
<td>0,594</td>
</tr>
<tr>
<td>“Conflict points” 1 and 2</td>
<td>32</td>
<td>0,500</td>
</tr>
</tbody>
</table>

Table 5.3 Example of a station’s complexity.

The conflict rate $\phi_0/\phi$ can thus be an expression of how well it has been intended to avoid unnecessary conflicts [3]. Conflicts of the type O, I and U can of course not be avoided, where several traffic flows meet, but the unnecessary conflicts can be limited or eliminated by building crossings out of grade. In addition, it is possible to introduce alternative routes in order to reduce the complexity index of the station. For the station shown in figure 5.8 it will however result in the need for one or several extra platform tracks, so that passing and/or overtaking can take place, or that the principle of driving on the right track is as a minimum deviated from (so that routes a and d will not be conflicting).

Since not all routes occur with the same frequency, it is rational to determine the stations conflict rate by weighing the individual conflicting routes according to their occurrence.
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<table>
<thead>
<tr>
<th>Occurrence of the routes</th>
<th>a (10%)</th>
<th>b (10%)</th>
<th>c (20%)</th>
<th>d (15%)</th>
<th>e (5%)</th>
<th>f (5%)</th>
<th>g (20%)</th>
<th>h (15%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a = 10%</td>
<td>0,0100</td>
<td>0,0100</td>
<td>0,0200</td>
<td>0,0150</td>
<td>0,0050</td>
<td>0,0050</td>
<td>0,0200</td>
<td>0,0150</td>
</tr>
<tr>
<td>b = 10%</td>
<td>0,0100</td>
<td>0,0100</td>
<td>0,0200</td>
<td>0,0150</td>
<td>0,0050</td>
<td>0,0200</td>
<td>0,0150</td>
<td></td>
</tr>
<tr>
<td>c = 20%</td>
<td>0,0200</td>
<td>0,0200</td>
<td>0,0400</td>
<td>0,0300</td>
<td>0,0100</td>
<td>0,0100</td>
<td>0,0400</td>
<td>0,0300</td>
</tr>
<tr>
<td>d = 15%</td>
<td>0,0150</td>
<td>0,0150</td>
<td>0,0300</td>
<td>0,0225</td>
<td>0,0075</td>
<td>0,0075</td>
<td>0,0300</td>
<td>0,0225</td>
</tr>
<tr>
<td>e = 5%</td>
<td>0,0050</td>
<td>0,0050</td>
<td>0,0100</td>
<td>0,0075</td>
<td>0,0025</td>
<td>0,0025</td>
<td>0,0100</td>
<td>0,0075</td>
</tr>
<tr>
<td>f = 5%</td>
<td>0,0050</td>
<td>0,0050</td>
<td>0,0100</td>
<td>0,0075</td>
<td>0,0025</td>
<td>0,0025</td>
<td>0,0100</td>
<td>0,0075</td>
</tr>
<tr>
<td>g = 20%</td>
<td>0,0200</td>
<td>0,0200</td>
<td>0,0400</td>
<td>0,0300</td>
<td>0,0100</td>
<td>0,0100</td>
<td>0,0400</td>
<td>0,0300</td>
</tr>
<tr>
<td>h = 15%</td>
<td>0,0150</td>
<td>0,0150</td>
<td>0,0300</td>
<td>0,0225</td>
<td>0,0075</td>
<td>0,0075</td>
<td>0,0300</td>
<td>0,0225</td>
</tr>
</tbody>
</table>

Table 5.4 Example of the frequency $P_k$ of conflicting routes (in bold-faced types).

If the conflict rate $\varphi_0 / \varphi$ is calculated according to the principle of weighting the individual conflicting routes according to their occurrence the following result is obtained:

<table>
<thead>
<tr>
<th>Crossings out of grade in figure 5.8</th>
<th>Complexity of the station $\varphi = P_k / P_\Sigma$</th>
<th>Conflict rate $\varphi_0 / \varphi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>0,715 = $\varphi_0$</td>
<td>1,00</td>
</tr>
<tr>
<td>“Conflict point” 1</td>
<td>0,645</td>
<td>1,11</td>
</tr>
<tr>
<td>“Conflict point” 2</td>
<td>0,590</td>
<td>1,21</td>
</tr>
<tr>
<td>“Conflict points” 1 and 2</td>
<td>0,535</td>
<td>1,34</td>
</tr>
</tbody>
</table>

Table 5.5 Example of a station’s complexity under consideration to the train routes occurrence.

Apparently it seems that if it for financial reasons is only possible to have one crossing out of grade, it will have the biggest effect if conflict point 2 is eliminated. However, to be absolutely certain it must be assessed for how long the individual routes will be occupied each time.
a time passes. By multiplying\(^6\) these block times to the frequencies of the individual route conflicts, you can evaluate which of the 2 conflict points that will be most inconvenient by calculating the conflict rate \(\phi_0/\phi\) in the same way as in table 5.3 and table 5.5.

### 5.3 New construction / extension

If the capacity reserve on a double track line is unacceptably small, it can be increased by establishing a new line in a new corridor, so that the existing railway is released. Alternatively, the existing railway can be extended with 1 or 2 extra main lines. From an operational point of view the 2 possibilities, new construction or extension, includes different aspects.

In case of an extension, 1 or 2 extra main lines can be built according to the need. If you decide to extend the railway with an extra main line, the expended railway can be operated according to the following 3 principles.

1. The extra main line can be used as a long overtaking track between the stations, so that it is alternately available for the 2 directions of travel, as shown in figure 5.9.

2. One direction operation on the entire line with 2 tracks in one direction and 1 track in the other direction, so that the rush hour traffic is considered with the extra track.

3. The different train categories are separated, so that e.g. freight trains (and/or high-speed trains) only use the extra main line on the same conditions as on a single track line (however, with the possibility of using one of the other 2 main lines in case of operation troubles), while the other traffic uses the existing track.

If a 3-track line is equipped with a single-line block system with directional control, it will be possible to operate the line optimally according to all 3 principles as needed.

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\(^6\) This can be done by multiplying either vertically or horizontally in a table like table 5.4
Capacity on double track lines

In Denmark, there are no 3-track lines, but they are e.g. found in Germany, where the above principle is also used. In road traffic, the principle No. 1 with long overtaking tracks are known from 3-track roads. On these, the direction of travel changes in connection with crossroads/side roads, where there are refuges/ghost islands that take up a lot of space. The road between Ringsted and Roskilde is an example of this. Principle No. 2 is also used in road traffic on motorways in connection with roadworks, where it is not possible to maintain operation of 4 lanes. Here the primary direction will therefore be assigned 2 lanes, whereas the secondary direction only has one lane. In this way, you obtain the best utilization of the available capacity, as the primary direction can be changed during the day, so that it is identical to the rush hour traffic. Principle No. 3 is also known from road traffic. On certain stretches of the road, it can be difficult for certain types of vehicles to drive. For instance, it will be particularly difficult for heavy vehicles to drive on a steep slope at the same speed as passenger cars. Therefore, the main roads are often fitted with a slow lane which can be used by heavy vehicles so that they do not reduce the passability of the other vehicles. Equally, you will often in the big cities find lanes that are reserved for special types of vehicles, e.g. bus lanes. In certain cases they are only established for one direction of travel, if the passability is especially critical for this direction.

In case of an extension to 4 parallel tracks it will be convenient to use one direction operation on all tracks (2 in each direction). The traffic can be further separated, so that apart from the one direction operation the traffic is separated, so that certain tracks are reserved for trains with a high/low average speed. In this way, an extension with 2 new tracks will more than double the capacity, as all the tracks will be operated with homogeneous traffic. In the same way an extension with 1 extra track will not only give the sum of the capacity of the original double track line summed up with the capacity of a single track line, as there will be a derived effect for the existing track due to a more homogeneous traffic.

If a new construction solution is chosen, it can in principle be considered as totally independent of the existing infrastructure. The only contact between the existing and the new infrastructure will be at
the link circuits. The new construction thus provides a possibility to create a new market, provided halt(s) are introduced on the new railway.

In case of operation troubles, it will be possible to divert traffic which is throughgoing between the 2 link circuits. This will in particular favour this solution in situations, where e.g. a derailment makes it necessary to block all tracks temporarily at the scene of the accident, as it will then be possible to operate some of the traffic on the alternative line, whereas all railway traffic would have to be suspended temporarily in case of 4 parallel tracks. On the other hand, if one track is blocked, an extension will be more convenient than a new construction, as 3 parallel tracks as earlier described have a bigger capacity than what can be obtained by means of a single and the double track line.

References


Capacity on double track lines
Chapter 6: Capacity on single track lines

It is often more complicated to calculate the capacity on single tracks than on double tracks. On single track, apart from the overtaking issue you must also take into consideration the crossing issue. An unfortunate dispatching of the traffic can even mean that the system is locked (= deadlock), so that no trains can operate the line.

Single track lines can of course only operate the traffic between 2 stations in one direction at a time. The distance between the crossing stations is therefore very important with respect to the capacity that can be obtained on the line, if you compare it with how important the mutual distance between the overtaking stations are for the line capacity on a double track line. However, the entry and exit conditions of the crossing stations are also essential, as it can contribute to reducing the crossing time \( t_x \), so that the traffic flows on the single track line can change direction faster. Parallel movement (cf. section 6.2.1) from 2 of the station’s railway lines will thus contribute to an optimum operation.

Figure 6.1 shows how a single track line can be designed with caution signals (Presignal), entrance signals, exit signals and pre-exit signals.

![Figure 6.1 Schematic track and signal diagram for a single track line.](image)

On the line between the 2 stations there can also be placed a number of block signals whose only purpose is to increase the capacity in situations, where the line is operated by 2 trains going in the same direction just after each other. In this way, a closer headway can be obtained.

6.1 Dimensioning line sections

The dimensioned line section (the infrastructure between 2 crossing stations) on single track lines will normally be the longest line section of the analysis area. Exceptions may however occur, e.g. if the permitted maximum line speed varies a lot. Furthermore, convoy
Capacity on single track lines

operation\(^1\) on a long line section can result in that the bottleneck is "transferred" to a shorter line section, where the traffic is bundled just as well.

If it is assumed that the line section shown in figure 6.1 gives the biggest capacity limitation for the entire single track line, the dimensioning average headway time \(t_{h,a}\) can be found by observing the individual headways of the line section between stations A and B. If the line is only operated by one train class, there can be 4 different headways. If the direction of travel from station A to station B is called index A and the opposite direction index B, the dimensioning average headway time \(t_{h,m}\) can be determined as follows:

**Formula 6.1**

\[
t_{h,a} = \frac{n_{AA} \cdot t_{h,AA} + n_{AB} \cdot t_{h,AB} + n_{BA} \cdot t_{h,BA} + n_{BB} \cdot t_{h,BB}}{n_{AA} + n_{AB} + n_{BA} + n_{BB}}
\]

Where: \(t_{h,ij}\) is the headway time for a train running in direction \(i\) followed by running in direction \(j\)

\(n_{ij}\) is the number of headways, where trains running in direction \(i\) are followed by a train running in direction \(j\)

If the trains on the line have different driving properties (primarily average speed) the above expression must be generalised to include \(k\) train categories, which is done below:

**Formula 6.2**

\[
t_{h,a} = \frac{\sum_{(x,y)=(1,1)}^{(k,k)} (n_{AA,xy} \cdot t_{h,AA,xy} + n_{AB,xy} \cdot t_{h,AB,xy} + n_{BA,xy} \cdot t_{h,BA,xy} + n_{BB,xy} \cdot t_{h,BB,xy})}{\sum_{(x,y)=(1,1)}^{(k,k)} (n_{AA} + n_{AB} + n_{BA} + n_{BB})}
\]

Where: \(x\) is the train class for the train in front

\(y\) is the train class for the train behind

If the above expression is compared to the corresponding expression for double track lines, it is seen that there are four times as many headway combination possibilities, each of which contributing to the dimensioning average headway time \(t_{h,a}\). The above expression requires that interfaces to crossing stations and other line sections do

---

\(^1\) Several trains running in the same direction just after each other at a single track line before the traffic flow is turned around.
not give rise to capacity limitations which often will be difficult to fulfil!!
- Therefore the size of the individual headway times will not only
depend on the headway in question (direction of travel and train class).
Depending on how the timetable is constructed, there may arise
situations, where 2 different minimum headway times can be found on
the dimensioning line section despite it being the same headway.
Figure 6.2 shows how a headway time \( t_{h,BC,1} \) must be extended
because of a capacity limitation in the neighbouring line section.
Experience from UIC [5] shows that when calculating the practical
capacity \( K_f \) has to be adjusted in order to the number \( z \) of line section
on the single track line in the following way:

\[
K_f = \frac{\Delta T}{t_{h,a} + t_b + z \cdot 0,25\text{min/tog}}
\]

Where: \( t_b \) is the buffer time

To include an extra crossing station on a single track line it will
therefore according to formula 6.3 not necessarily result in an increase
in the practical capacity \( K_f \). If the new crossing station is placed on a
not dimensioning line section, the practical capacity of the entire single
track line will decrease according to formula 6.3 which of course
contrary to common sense, as an extra crossing station other things
being equal must result in better punctuality, as planned crossings can
be moved in case of troubles in the operation, thereby giving a better
operation. Formula 6.3 must therefore be used with precaution.

When comparing a short and a long single track line, where line
sections are roughly of the same length, the philosophy in formula 6.3
seems more correct. That is because the practical capacity on a long
single track line is smaller than on an equivalent short line, since
operation troubles tend to spread more easily on single track lines.

6.2 Crossing station

In figure 6.3 the capacity on line sections A-B and B-C is fully utilized
with the chosen timetable pattern. Thus, the unbroken lines make up a
cycle consisting of \( N = 5 \) trains that are repeated in the period just
before and after, respectively.
Capacity on single track lines

By relating the average headway time $t_{h,a}$ to one of the stations (e.g. the station boarder at station C against station B) it can be determined by summing up the individual contributions, as shown in figure 6.3. No matter where on the line section, the average headway time will be the same, as we are dealing with a cyclic traffic operation. The size of the individual contribution may vary, but the sum will remain the same. If station C is used as reference, the contributions from $t_{h,CB}$ will be very big, whereas contributions from $t_{h,BC}$ are very small. However, if station B is chosen as reference, the opposite will be the case. With station C as reference, $t_{h,BC}$ and $t_{h,CB}$ are written in the following way:

**Formula 6.4**  
$t_{h,BC} = t_{x,C}$

**Formula 6.5**  
$t_{h,CB} = t_{k,C} + t_{k,B} + t_{x,B}$

Figure 6.2 Graphical timetable for single track line.
Where: $t_{x,C}$ and $t_{x,B}$ is the crossing time on stations C and B respectively, cf. formula 6.6

- $t_{k,C}$ is the running time from station C to station B
- $t_{k,B}$ is the running time from station B to station C

Running times $t_{k,B}$ and $t_{k,C}$ can be determined as indicated for inhomogeneous traffic on double track lines.

To create sufficient time for a crossing to take place, the necessary dwell time (named crossing time $t_x$) on the crossing stations will often be longer than the dwell times $t_h$, which have exclusively been determined on grounds of the boarding and exit conditions. If you furthermore use an operation strategy where several trains are going in the same direction on the single track line, the train that is brought to stop and is to be crossed by several trains, will be exposed to a very long dwell time. With figure 6.3 as an example, the crossing time $t_{x,B}$ on station B could be expressed in the following way:

**Formula 6.6**

$$t_{x,B} = t_{x,AB} + t_{x,BB} + t_{x,CB}$$

Where:
- $t_{x,AB}$ is the time required from the train that comes from station A has arrived at station B until the train from station B can leave towards station A
- $t_{x,BB}$ is the time required between 2 trains leaving station B towards station A
- $t_{x,CB}$ is the time required from the train that comes from station C has arrived at station B until the train from station B can depart towards station C

For headways where the trains are going in the same direction, the individual headways can be calculated in the same way as for double tracks. And for this kind of headways, the block signals will, as earlier mentioned, have a capacity increasing effect.

### 6.2.1 Parallel movement

For headways, where the trains are going in opposite directions, the crossing time $t_x$ will partly depend on the technical character of the stations and partly on the driving pattern of the train.

To create a good traffic operation the crossing station can be designed in such a way that it can handle parallel movement.
To allow for parallel movement, it is necessary to create a sufficient safety distance \( F \) behind the pre-exit signal. It can be done in 2 ways. Either as shown in figure 6.3 by means of a dead-end track or by placing the pre-exit signal in the necessary safety distance \( S_s \) from the fouling point. The size of the safety distance \( S_s \) will depend on the speed \( v_{\text{ent}} \) at which the train is allowed to enter. The speeds are shown in the following table:

<table>
<thead>
<tr>
<th>Entry speed ( v_{\text{ent}} ) [km/h]</th>
<th>Safety distance ( S_s ) [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \leq 40 )</td>
<td>50</td>
</tr>
<tr>
<td>60</td>
<td>100</td>
</tr>
<tr>
<td>( \geq 80 )</td>
<td>150</td>
</tr>
</tbody>
</table>

Table 6.1 Necessary safety distance \( S_s \) as function of the entry speed \( v_{\text{ent}} \).

If the daily operation is as shown in figure 6.3, where 2 trains at approximately the same distance are approaching a crossing station from opposite directions, it will be possible for the trains to cross each other, without having to stop completely, provided the length \( L_k \) of the crossing track is sufficient. It is because the trains will be operated at the permitted entry speed \( v_{\text{ent}} \) and start their gradual braking, so that it is possible for them to stop in front of the pre-exit signal. When train 1 has released its entry route, train 2 will get the signal “go” in the pre-exit signal, and it can therefore accelerate without having stopped completely, provided the entry takes place at approximately the same time from both sides of the station. The train’s minimum speed \( v_{\text{min}} \) during the crossing can be determined by considering the braking distance \( S_b \) required by the minimum speed \( v_{\text{min}} \).

Formula 6.7 \[ S_b = L_k - L \]
Where: \( L \) is the length of the train

The braking distance \( S_b \) can then be converted to minimum speed \( v_{\text{min}} \) based on the knowledge about the correlation. An IC3-train with an entry speed \( v_{\text{ind}} = 80 \text{ km/h} \) thus requires a braking distance of approx. 300 m. If the signal watching time and the time for setting up the train route are ignored, it thus requires that the distance between the fouling points is approx. 650 m, if the length of the train is approx. 200 m, and the station does not have dead-end track. Parallel movement is especially convenient if the crossing station is only a regulating station with no exchange of passengers. In this way the two trains do not have to stop, as they only need to reduce the speed temporarily. This causes a running time extension \( \Delta t_x \), which can be calculated in the following way:

**Formula 6.8**

\[
\Delta t_x = t_{\text{dec}} + t_k + t_{\text{acc}} - t_0
\]

Where:  
- \( t_{\text{dec}} \) is the time the train decelerates
- \( t_k \) is the running time that the train drives at the constantly reduced speed
- \( t_{\text{acc}} \) is the time the train accelerates
- \( t_0 \) is the running time that it would take the train to pass the line if the speed was not reduced

In a situation, where two IC3 trains driving at 40 km/h are to cross each other at a station that allows parallel movement at 80 km/h, the running time extension \( \Delta t_x \) can be determined in the following way, among other things by checking in tables showing the acceleration and retardation properties:

\[
\Delta t_x = (48 - 27)s + \frac{500 \text{ m}}{80 \text{ km/h}} \cdot 3.6 \text{ km/h m/s} + (108 - 35)s - \frac{(936 - 305)m + 500 \text{ m} + (2801 - 469)m}{140 \text{ km/h}} \cdot 3.6 \text{ km/h m/s}
\]

\( \Delta t_x = 27s \) (provided that the trains operates at a constant speed of 80 km/h on a section of 500 m)

A “moving” crossing will thus give the trains a travel time extension of approx. \( \frac{1}{2} \) minute if the crossing station offers optimum conditions and the trains arrive at the same time.
Capacity on single track lines

In figure 6.4 is shown the speed-distance curves of the trains in case of parallel movement at a station with a passing loop.

![Speed v [km/h]]

Figure 6.4 Crossing in motion with parallel movement.

At a crossing situation, where one train has to stop completely to pick up/drop passengers, this train should arrive to the crossing station first. With parallel movement the throughgoing train can obtain permission to pass, when the stopping train with its entire length L has passed the fouling point. Without parallel movement this cannot happen until the stopping train stands totally still\(^2\). If the throughgoing train is situated just before the caution signal (or the last block signal), when the route for throughgoing train is set, it is possible for the throughgoing train to drive without reducing the speed because of the crossing. Normally, the dwell time of the stopping train will not be extended due to the crossing, but in case of bad coordination of the arrival of the two trains it may have a longer stay at the station.

Without parallel movement the dwell time of the stopping train has to be at least 2 – 3 minutes, as the throughgoing trains can only pass the entrance signal, when the stopping train stands still. But in order to

\(^2\) This is an approximation. In fact, it is not the speed of the stopping train and its position which determine when the train route for the transit/throughgoing train can be set. It is a fixed time interval (e.g. 60 sec.) after the block has been occupied which empirically implies that the stopping train stands still.
obtain a high speed for the throughgoing train must be behind the caution signal, when the train route for the throughgoing train is set.

### 6.3 Operation patterns

As regards the capacity, a single track line is very sensitive to the operation patterns. The operation of the trains will thus mutually be very closely related. The possibilities of overtaking will be very limited, as crossings will be the most frequent change of the headway at the stations.

#### 6.3.1 Deadlock situations

On single track lines with a high utilization, the dispatcher must be very careful not to create a locked situation (deadlock), where the trains will accumulate on the line and at the intervening stations, so that the trains will have to back to proceed!! Such a deadlock situation will of course never occur, if the operation is performed according to the timetable. In situations with irregular traffic, e.g. if one of the tracks of a double line is completely blocked, it may however occur, if the dispatching is unfortunate.

It is very complicated to set up exact rules of dispatching to avoid deadlock situations, if the rules are not made very restrictive, and thereby exclude certain events that normally could occur without causing any problems [2]. Below is shown an example of a single track line operated close to the capacity level.

![Traffic on a single track line](image)

Figure 6.5 Traffic on a single track line.

If both train 1 and train 4 proceed one station, we have a deadlock situation, as train 1 and train 2 in this case will be locked by train 3 and train 4 (and the other way round). However, there are many other operation strategies that will prevent the deadlock from occurring, cf. figure 6.6.
Capacity on single track lines

Train movements:

Figure 6.6 Search tree showing possible train movements.

Each arrow indicates a train movement, where the train number is stated at the arrowhead. A train movement is defined as the transport of a train from one station to the neighbour station. It can for instance be seen that train movements →1→4 and →4→1 are excluded, as they would lead to deadlocks.

Figure 6.6 only show the combination possibilities for the 3 first train movements. Thus, there is a total of 26 combination possibilities for the 3 first train movements that are possible. Consequently, relatively many of the $4^3 = 64$ combination possibilities have been excluded. Apart from train movements, which will lead to deadlocks, also those train movements resulting in a situation where all the platform tracks at a station are occupied, have been excluded. The train also “disappears” from the system, when it arrives at the transition station as it is expected to have unlimited capacity, as the neighbouring line is double tracked.

The complete search tree for the situation shown in figure 6.6 means that each combination possibility includes 10 train movements as train 1, train 2, train 3 and train 4 must proceed 3, 2, 2 and 3 stations, respectively, in order to pass the bottlenecks (= arrived at the transition station to double track). Therefore, the search tree includes several thousand combination possibilities that are possible in practice. However, figure 6.6 is rather simplified as compared to real life, as new trains will continuously enter the single track line. In principle, the search tree can therefore be unendless. The aim of the search tree is not to find the optimum order of the train movements, but only to exclude those leading to deadlocks. However, you will not realize this until the search tree has been finished and there are no trains left in the system.

It will be difficult for a dispatcher to foresee the “complete” search tree, and he will therefore use more simplified rules to avoid deadlocks, even though the rules may result in some trains having to wait longer than actually necessary. In the same way, the simulation models must contain simplified rules of dispatching ensuring that deadlocks will never occur.
6.3.2 Convoy operation

As it was shown in the chapter on the capacity on double track lines, bundling of the trains based on their average speed will increase the capacity, and on single track lines when organizing the trains in the same direction to run just after each other (convoy operation) will result in increased capacity.

The time $t_{cyk}$ it takes to operate a convoy consisting of $n$ trains driving in one direction followed by $n$ trains driving in the other direction can be determined in this way:

Formula 6.9  \[ t_{cyk} = 2 \cdot t_{k,AB} + 2 \cdot (n - 1) \cdot t_h \]  

[1][4]

Where: $t_{k,AB}$ is the running time from station A to station B (or reversed, as they are supposed to be identical)

$n$ is the number of trains driving in the same direction just after each other

$t_h$ is the headway time between 2 trains driving in the same direction

When the time $t_{cyk}$ it takes to operate a cycle is known, the maximum capacity $K_{max}$ can be determined in the following way:

Formula 6.10  \[ K_{max} = \frac{2n}{t_{cyk}} \]

If formula 6.9 is inserted into formula 6.10, we get:

Formula 6.11  \[ K_{max} = \frac{2n}{2 \cdot t_{k,AB} + 2 \cdot (n - 1) \cdot t_h} = \frac{n}{t_{k,AB} + (n - 1) \cdot t_h} \]

\[ \frac{1}{t_{k,AB}} + \frac{1 - \frac{1}{n}}{t_h} = \frac{1}{t_{k,AB}} + \frac{1}{n} \left( t_{k,AB} - t_h \right) \]

It appears from the above that provided $t_{k,AB} > t_h$, the capacity $K_{max}$ will increase by increasing convoy size $n$. As this will normally be the case, a convoy operation of the trains will improve the capacity.
Capacity on single track lines

However, this convoy operation will unfortunately result in the maximum waiting time $t_{v,\text{max}}$ between two trains driving in the same direction will increase for increasing $n$, as appears from the following expression:

$$t_{w,\text{max}} = 2 \cdot t_{k,AB} + (n - 1) \cdot t_h$$  \[1\][4]

In the below table, the capacity $K_{\text{max}}$ and the maximum waiting time $t_{w,\text{max}}$ have been calculated for different combinations of the convoy size $n$ and the running time $t_{k,AB}$ between two crossing stations, where the headway time $t_h$ is assumed to be 3 min.

<table>
<thead>
<tr>
<th>Running time $t_{k,AB}$ [min]</th>
<th>n=1</th>
<th>n=2</th>
<th>n=3</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>$K_{\text{max}}$ [trains/h]</td>
<td>10</td>
<td>13.3</td>
</tr>
<tr>
<td></td>
<td>$t_{w,\text{max}}$ [min]</td>
<td>12</td>
<td>15</td>
</tr>
<tr>
<td>9</td>
<td>$K_{\text{max}}$ [trains/h]</td>
<td>6.7</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>$t_{w,\text{max}}$ [min]</td>
<td>18</td>
<td>21</td>
</tr>
<tr>
<td>12</td>
<td>$K_{\text{max}}$ [trains/h]</td>
<td>5</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>$t_{w,\text{max}}$ [min]</td>
<td>24</td>
<td>27</td>
</tr>
</tbody>
</table>

Table 6.2 Capacity $K_{\text{max}}$ and maximum waiting time $t_{w,\text{max}}$ for different combinations of section driving times $t_{k,AB}$ and convoy sizes $n$.

As appears from the table, the capacity $K_{\text{max}}$ grows degreisively, both for increasing convoy size $n$ and increasing running time $t_{k,AB}$. On the other hand, the maximum waiting time between two trains driving in the same direction is constantly growing for both increasing convoy size $n$ and increasing running time $t_{k,AB}$

### 6.4 Capacity increasing measures

To obtain a high capacity on single track lines, a number of conditions regarding the infrastructure, which can increase the capacity if they are designed properly, must be taken into considerations. These can be divided into the following:

- Increased line speed
- Establishment of several crossing stations
- Introduction of parallel movement
- Extension of tracks at stations
- Increased speed at the switches for the deflecting section
By increasing the line speed the running time between the crossing stations is reduced. This is particularly useful for lines with a long distance between the crossing stations, and on lines where headways with 2 trains driving just after each other in the same direction, do not occur. This is shown in table 6.2

By establishing many crossing stations on a single track line it is more likely that the traffic flows in the opposite direction can meet at the crossing stations, without disturbing the traffic considerably. In situations where the trains run on schedule, the timetable will normally be constructed in such a way that the crossing times are limited as much as possible. In case of operation troubles, it will however in certain situations be expedient to move the crossing to the neighbour station. It is particularly the case, if the distance between the crossing stations is relatively short. By introducing parallel movement at the crossing stations, the crossing time $t_x$ can be reduced by a couple of minutes. It should however be observed that the reduction is only possible if the trains actually arrive more or less at the same time. Apart from the technical conditions, the timetable must therefore be constructed in such a way that the crossing times are minimized if the wanted effect is to be obtained.

With relatively long overtaking/crossing tracks it will be possible to carry out a crossing without the trains having to stop. Long passing tracks will also allow for long freight trains to overtakes/cross at the station. In this way the single track line will be more flexible, as it in case of operation troubles is possible to move the crossing of a long freight train.

An increased entry speed $v_{ent}$ will allow for a smaller headway time $t_h$. The permitted speed in the deflecting section of the switch will depend on “the inclination”\(^3\), as appears from table 6.3:

\(^3\) “Inclination” is to be understood as the relation between the sides of the triangle that is formed between the deflecting and main track of the switch as shown below:
Capacity on single track lines

<table>
<thead>
<tr>
<th>Inclination</th>
<th>Radius [m]</th>
<th>Length [m]</th>
<th>Speed on deflecting section [km/h]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:5,45 (symmetrical)</td>
<td>215</td>
<td>22,09</td>
<td>40</td>
</tr>
<tr>
<td>1:7,5</td>
<td>190</td>
<td>27,30</td>
<td>40</td>
</tr>
<tr>
<td>1:9</td>
<td>190</td>
<td>27,30</td>
<td>40</td>
</tr>
<tr>
<td>1:9</td>
<td>300</td>
<td>33,01</td>
<td>50</td>
</tr>
<tr>
<td>1:11</td>
<td>330</td>
<td>34,02</td>
<td>50</td>
</tr>
<tr>
<td>1:12</td>
<td>500</td>
<td>40,21</td>
<td>60</td>
</tr>
<tr>
<td>1:14</td>
<td>500</td>
<td>41,05</td>
<td>60</td>
</tr>
<tr>
<td>1:19</td>
<td>1.200</td>
<td>60,95</td>
<td>100</td>
</tr>
<tr>
<td>1:19 (symmetrical)</td>
<td>2.400</td>
<td>60,95</td>
<td>120</td>
</tr>
<tr>
<td>1:26,5</td>
<td>2.500</td>
<td>93,49</td>
<td>130</td>
</tr>
</tbody>
</table>

Table 6.3 Maximum permitted speed through deviating track at switches [3].

It is thus possible to obtain speeds of up to 130 km/h in connection with overtaking/crossings of trains using the overtaking/crossing track. These high speeds however require relatively long switches meaning that the switch zone will be longer. This means that the running time through the switch will be extended with maximum 1 second, which will definitely be compensated by a running time reduction of approximately ½ minute, if e.g. the entry speed over a distance of 1.000 m can be increased from 60 km/h to 120 km/h on average.

**References**


Chapter 7: Timetabling

Constructing a timetable can be described as making a big puzzle without any picture on the pieces meaning that there will be several correct combination possibilities. Therefore, it will therefore be impossible to find the optimum timetable, as the optimization can be based on various criteria, e.g. the fastest running times or a regular interval timetable. However, the optimization is often based on several different and opposing considerations, as one criterion is not enough to make a good timetable.

Normally, the following quality parameters will be part of a timetable evaluation [4]:

- **High punctuality**
  The ability of the train to run on schedule is important for the traveller as it enables him to make other connections and observe scheduled appointment times, if any.

- **Direct connections**
  The number of transfers must be limited, as they are considered a nuisance, in particular when travel intensities are big.

- **Good transfer conditions**
  When transfers cannot be avoided, the transfer time to connecting trains (and maybe also to important bus lines) must be strictly adapted.

  In case of transfer it is furthermore desirable if the connecting trains can be caught from the opposite side of the platform in order to reduce the walking distance and thereby also the transfer time.

- **Regular departure schedule**
  Regular interval timetables and constant use of the same platform track also makes it easier for the passengers to remember the timetable.

- **Short travel time**
  The passengers wish a high travel speed and a limited number of stops during the trip. Therefore, it is necessary with several train systems on the lines where this is possible, so that certain trains do not stop at the less used stations.

---

1 Regular interval timetable for all similar train classes
2 Normally the bus lines will be adapted to the timetable of the railway and not the other way around, since there are more limitations in scheduling railway traffic
Timetabling

- High departure frequency
  Is important due to the competitiveness with alternative transport modes.

Several of the points will be directly conflicting. Thus, many direct connections and a high departure frequency mean a high utilization ratio of the infrastructure, but an intensive utilization of the infrastructure increases the risk of poor punctuality. Short transfer times means that in many situations the planned connection will not be possible, or you have to wait for the connection. In the latter, the travel time is extended “unnecessarily” for all the travellers who have not made transfers. However, if you do not have to wait for the delayed train, all the transboarding passengers suffer a substantial travel time extension equivalent to the frequency on the route to which they make the transfer. With a high departure frequency the operation speeds of the trains must/will adapt to each other, provided the traffic intensity is close to the capacity level. This will therefore increase the travel time for trains that under free conditions could run at higher speeds, and the risk of poor punctuality is also increased.

A regular interval timetable, where all the trains are assigned a fixed departure interval, will often result in bad capacity utilization, as it is not possible to bundle trains with same driving behaviours. This situation is shown in the left part of figure 7.1.

![Figure 7.1 Different optimisation of the timetable.](image-url)
The situation reproduced in the right part of figure 7.1 shows that by letting trains with approximately the same speed drive just after each other, a less time consuming traffic operation (because $t_{s,\text{pas}} > t_{s,\text{K}}$) can be obtained. However, this type of timetable is not optimal for the customers, since their wish for a fixed departure interval cannot be fully met. This is due to the fact that normally the train category and the average operation speed is correlated, so that trains in demand by the same customer group will depart just after each other.

In case of a specific timetable, the needs of the customers on a line with a very high utilization should also be taken into consideration, and in practice it will therefore be difficult to operate the capacity-optimized timetable, if the traffic on the line is not homogeneous. Therefore, the solution will often be a compromise between the 2 solutions shown in figure 7.1, which can e.g. be obtained by letting 2 regional trains with different destinations drive just after each other, whereas the time interval between the long-distance trains is not changed.

### 7.1 The planning process in case of timetabling

The time horizon in connection with planning of the necessary/possible train supply is very long. The set of feasible solutions of possible timetables will be reduced when the date where the final timetable comes into force is approaching.

![Diagram](image)

**Figure 7.2** Limitations influence on the set of feasible solutions at train scheduling [6].

As appears from figure 7.2, in the beginning it will only be the available infrastructure (e.g. single track or double track) and the available rolling stock that form the basis of the possible timetable. There will thus be several overall capacity limitations that reduce the set of feasible solutions. The product needs will limit the set of feasible
solutions to include only the possible timetables that are supposed to cover the customers’ demand. In principle, the product needs could fall outside the area covered by the set of feasible solutions, but in the light of earlier experience, and what is possible in practice, the individual train operators have already made an internal evaluation when presenting the product needs.

The product wishes are compiled to a plan of operation that describes which train products are to operate the individual routes, and which stopping pattern and departure frequency are wanted. By coordinating the individual products in the plan of operation, we get a candidate timetable that will form the basis of the final timetable. Thus the plan of operation does not contain information about the wanted order of departure of the individual trains.

If the various train operators cooperate during this phase, the customers will usually obtain the best result, among other things because of a good capacity utilization. The cooperation between train operators whose primary objective is the transportation of international travellers and local passengers, respectively, is for instance to create the best customer conditions. In this way the transfer times between the 2 systems can also be minimized. Furthermore, it allows passengers who travel to the big cities along the line, where the international trains should pick up and drop passengers, to use these trains as a through service. This is not only customer-friendly, but also increases the capacity, as the number of proper regional trains can thereby be reduced, as the international trains will have to cover part of the need for permanent connections which is demanded by the local travellers using the line.

When preparing the final timetable, the departure times of the individual train products are made more specific. Usually, the existing timetable is used as a starting point, as it will normally represent a set of acceptable conditions under the given prerequisites. During the planning process the initial phases can therefore often be omitted or be very short, as the new timetable will primarily be based on previous experience. Therefore, it will be intended to fit in an extra train in a new timetable, e.g. by “displacing” a couple of trains a few minutes, so that the original timetable does not have to be broken into pieces.

The entire planning process in connection with the establishment of the supply of trains will be supported by capacity studies. Particularly during the initial phases, these will be carried out by means of computerized simulation programmes which try to simulate the complex situations that are expected to arise with the given combination of infrastructure and plan of operation/timetable. As a relatively new matter in Denmark, the customers are officially consulted during the entire planning process. In recent years a number of commuter organizations has arisen during the privatization of the Danish State Railways (DSB), and they have the possibility to influence
both the product wishes and the final timetable. Also major investments in infrastructure are discussed publicly so that also the infrastructural conditions can be adapted as much as possible to the customers’ and the society’s wishes and needs [7].

### 7.2 The dimensions of travel time

The travel time of the journey carried out can be divided into a number of time phases. For train travellers, some of these will feel more troublesome than others. The transfer time between 2 trains will for instance feel very bothering, but if it is reduced too much, you will often miss the connection to the train, or it may bother the traffic on the secondary railways (or the main railway line). Therefore, the size of the transfer time is a dilemma. In the same way, time elements such as running time supplement, dwell time etc., will form part of the total travel time, even though the train passengers may think of them as queuing time. These time elements are established based on knowledge about the capacity, the operation punctuality, etc. of the individual lines.

In figure 7.3 is shown which principle contributions the realized travel time consists of.

*The minimum running time* is thus an expression of the travel time that will be possible provided a train passenger can travel under optimum conditions³ from the start station to the destination station.

*The running time supplement* ensures that expected operation troubles and infrastructural conditions will not necessarily delay the train service as compared to the existing timetable (cf. section 7.2.2). In case of important temporary infrastructure troubles, the trains can be assigned *extra supplements* in the planned running time. In this way, the *shortest running time* is obtained.

Since trains are part of public transportation, the train travellers will often have to stop on the way to pick up/drop other passengers. These *dwell times* will thus form part of the *shortest travel time*.

As a minimum, the dwell time will be an expression of the time allowing passengers to get in and out of the train. This dwell time can be supplemented if it is necessary to await passengers from connecting trains (cf. figure 7.4) or it may be extended because of the crossing time required on single track lines.

---

³ This means without any stops, transfers and without taking into considerations other trains.
If it is not possible to obtain a direct train connection, the *fundamental travel time* must include the necessary *transfer times*. In order to ensure as far as possible the connection between two trains in case of transfer, not only the minimum transfer time must be taken into account, but also the *planned waiting time* (cf. section 7.2.1).

All the above contributions thus make up the *planned travel time* that corresponds to the time used which is found when consulting the timetable. If the actual trip *deviates from the timetable*, the *realized travel time* will be extended as compared to the planned travel time, as the train must not depart before the time stated in the timetable. If the trip requires one or several transfers, a timetable deviation can lead to an important travel time extension, as the connection to a connecting train can be lost.

### 7.2.1 Connection conditions

At junction stations where several train routes meet, there will be a special need to coordinate the train service. Figure 7.4 thus shows how it is possible to create a transition between two trains in both directions by extending the dwell time.
A train that has to await passengers from another train is assigned a synchronization time to allow for the necessary transfer time. The planned dwell time expresses how long time a train must stop to allow for transfer to the train under normal circumstances. Due to operation troubles, the dwell times are assigned a supplement that can be directly related to the operational conditions (including the departure procedure). This supplement can also be an expression of the capacity limitations of the infrastructure, e.g. in connection with single track operation. For the transit passengers the waiting time can be quite long, and therefore the travel flows at a junction should be evaluated carefully in order to minimize the total waiting time per passenger. Thus, there should not for any price be obtained good connection conditions for small passenger intensities, as it will often have a negative impact on the travel time of the primary flow.

By letting the train with low priority (train 1) arrive a few minutes before the train with high priority (train 2), the synchronization time for train 2 can be reduced or even eliminated. If the difference in the arrival time is even bigger, the planned waiting time for train 2 can also be reduced/eliminated, whereby train 2 will not be influenced by the connection on the station in question. If the station is the end station of a secondary railway, the passengers in train 1 will not feel that the travel time is extended because of the ideal conditions for train 2 when the train is running on time, as there will not be transit passengers in train 1.
7.2.2 Running time supplement

The size of the queuing time $t_q$ can be used as a quality objective for how well the plan of operation and the infrastructure agree. The queuing time $t_q$ is defined as the extra running time the individual trains are imposed, because there are several trains in the system at the same time. Thus, all the trains on a given route have a minimum running time $t_{min}$ that includes the planned halts, if any. $t_{min}$ can be determined based on the physical driving behaviour of the trains (maximum speed, braking and acceleration), and the technical nature of the infrastructure (line speed and gradient). The minimum running time $t_{min}$ will normally never be found in practice, as the timetable must allow for a number of foreseen and unforeseen events which will make it impossible to go through the route in the minimum running time $t_{min}$. The timetable takes this into account in the form of running time supplements $t_{rts}$. As passenger trains must not leave before the time indicated in the timetable, the optimum running time $t_{opt}$ for a train on a given route will be:

Formula 7.1 $t_{opt} = t_{min} + t_{rts}$

The running time supplement can e.g. be determined as time supplements that depend on the length of the route and the nature of the infrastructure (single or double track) and the rolling stock (Intercity or regional trains). In Denmark, the size of the time supplements is as follows:

<table>
<thead>
<tr>
<th>Train class</th>
<th>Double track</th>
<th>Single track</th>
</tr>
</thead>
<tbody>
<tr>
<td>IC train</td>
<td>0,03 min/km</td>
<td>0,05 min/km</td>
</tr>
<tr>
<td>Regional train</td>
<td>0,05 min/km</td>
<td>0,08 min/km</td>
</tr>
</tbody>
</table>

Table 7.1 Running time supplements.

In capacity models it is often optional to include these running time supplements. If the supplements are included in the simulations, this is done by reducing the maximum speed $v_{max}$ of the train on a given route by a percentage equivalent to the size of the supplements. If the maximum speed $v_{max}$ of the train is higher than the permitted line speed $v_{pls}$, the latter is reduced corresponding to the size of the supplement. It is possible to define several punctuality points on a route, normally the major stations (e.g. halts for IC and EC trains or junction stations). In fact, it is between these punctuality points that a suitable strategy for the operation speed of the individual train is established in the plan of operation allowing for the necessary time...
supplements into consideration. If e.g. a regional train⁴ is operated on a 50 km double track line (between 2 punctuality points), where the average line speed is 120 km/h (the train can run faster), the maximum operation speed v of the train between the punctuality points is determined as follows:

\[ t_{opt} = \frac{50 \text{ km}}{120 \text{ km/h}} \cdot 60 \text{ min/h} + 0,05 \text{ min/km} \cdot 50 \text{ km} = 27\frac{1}{2} \text{ min} \]

\[ v = \frac{1}{t_{opt}} = \frac{50 \text{ km}}{27\frac{1}{2} \text{ min}} \cdot 60 \text{ min/h} = 109 \text{ km/h} \]

The above general reduction of speed can be characterized as the consequence of time supplements as it concerns a supplement that has to ensure that the train run on schedule even in case of minor operation troubles. In the same way you operate with a LA-supplement⁵ that ensures that there is time to reduce the speed on a short line section in connection with construction work. The size of this LA-supplement is found by reducing the speed of the train to 30 km/h on a line section of 100 m, after which it can accelerate to the original speed. For a regional train⁶ with an average speed of 120 km/h, the size of the LA-supplement t_{LA} can be determined as follows by means of information about the retardation and acceleration of the train:

Formula 7.2 \[ t_{LA} = t_{dec} + t_{30} + t_{acc} - t_0 \]

\[ t_{LA} = (52 - 13)s + \frac{100 \text{ m}}{30 \text{ km/h}} \cdot 3,6 \frac{\text{km/h}}{\text{m/s}} + (155 - 13)s\frac{(874 - 55) \text{ m} + 100 \text{ m} + (3,555 - 55) \text{ m}}{120 \text{ km/h}} \cdot 3,6 \frac{\text{km/h}}{\text{m/s}} \]

\[ t_{LA} = 60 \text{ s} = 1 \text{ min} \]

Where: \( t_{dec} \) is the time the train is braking
\( t_{30} \) is the time the train drives at a constant speed \( v = 30 \text{ km/h} \)
\( t_{acc} \) is the time the train accelerates

⁴ MZ locomotive with a length of train L = 192,3 m (equivalent to 7 wagons)
⁵ Running time supplement covering slow driving
⁶ MZ locomotive with a length of train L = 192,3 m (equivalent to 7 wagons)
Timetabling

\[ t_0 \] is the running time, if the train continues through “the LA-area” at max. speed

Normally, the LA supplements is assigned per 50 km, but on e.g. single track lines that have a high capacity utilization, the LA-supplements can be assigned more frequently in order to maintain a satisfactory punctuality. If the LA-supplement \( t_{LA} \) is bigger than the time supplements, this supplement is used as running time supplement \( t_{kt} \) in formula 7.1, otherwise the time supplements are used as running time supplement. In the calculated example, it will be the time supplements (2½ min) that have to be used as running time supplement as it is bigger than the LA-supplement (1 min).

The objective of the running time supplements is to create a high quality \( Q \) (punctuality) of the train service. Too large running time supplements will however be inconvenient as the travel time is hereby increased unnecessarily for the punctual trains. Therefore, it is a balancing act to determine the right size of these supplements. The present supplements have however been in force since 1977 which indicates that their size is reasonable under the circumstances up to now\(^7\). However, the size of the running time supplements should depend on the traffic load \( A \) and the homogeneity of the traffic composition, if you want a certain quality \( Q \). Big running time supplements can thus partly counteract the situation where an increased traffic load \( A \) will result in a lower traffic quality \( Q \). Another way to create quality in the train service is to operate with some buffer times \( t_b \) ensuring that the timetable will contain “buffers” between the trains, so that a small delay of one train will not spread to the following trains. Just as the running time supplements, the buffer times \( t_b \) are to be adapted carefully as they will reduce the capacity.

7.3 Punctuality

Punctuality is the quality parameters that travellers generally appreciate most [6]. With the railway liberalisation, the punctuality has been brought into focus as it is a measurable quality parameter that can be observed at the individual train operators. When the actual

\(^7\) In recent years, the traffic on some of the Danish main lines has become more inhomogeneous, primarily because the speed has been increased from 140 km/h to 180 km/h. However, freight trains and most regional trains will still run at a speed of approx. 100 km/h and 140 km/h, respectively, resulting in a bigger difference between the speeds of the trains. This should result in a larger running time supplement if you want the same quality \( Q \) in the train service. The rather poor acceleration properties of the IC3-trains at 180 km/h however mean that the LA-supplements will be rather big, which in principle can contribute to compensate for the more inhomogeneous traffic. However, this compensation is not obtained, if the trains have a high acceleration power at 180 km/h, and the big differences in speed on the same infrastructure should therefore be compensated in another way.
punctuality data are published, both the infrastructure owner, the travellers and operation companies that depend on freight transport evaluate whether the product is satisfactory with respect to the punctuality. Below are stated the objective for the Danish railways with respect to the various train products:

<table>
<thead>
<tr>
<th>Train product</th>
<th>Maximum delay</th>
<th>Punctual trains</th>
</tr>
</thead>
<tbody>
<tr>
<td>IC-, Express and</td>
<td>5 min</td>
<td>85%</td>
</tr>
<tr>
<td>Interregional trains</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Regional trains</td>
<td>2 min</td>
<td>85%</td>
</tr>
<tr>
<td>S-trains</td>
<td>2 min</td>
<td>95% (rush hours 94%)</td>
</tr>
<tr>
<td>Mail train</td>
<td>5 min</td>
<td>85%</td>
</tr>
<tr>
<td>Freight train</td>
<td>10 min</td>
<td>85%</td>
</tr>
</tbody>
</table>

Table 7.2 Object of quality concerning the punctuality on Danish railways [4].

Presently, the punctuality demands are being discussed in Denmark, and it is therefore not unthinkable that within a few years train operators will be met with much bigger demands in this field. With several different train operators on the Danish railways in the future, the operating safety and punctuality of the trains will be very important, as secondary delays that have arisen due to delays caused by competing train operator will be very unpopular. The punctuality can be determined according to several different methods. In Denmark, we normally only indicate the number of delayed trains and the size of these delays, whereas in Sweden you will often indicate the the possibility to regenerate the timetable (in Swedish Återställningsförmåga) which is defined in the following way:

Formula 7.3

\[ \hat{\Delta} = \frac{t_{\text{D,in}} - t_{\text{D,out}}}{t_{\text{D,out}}} \] [1]

Where: 
- \( t_{\text{D,in}} \) is the delay of the train when it enters the analysis area
- \( t_{\text{D,out}} \) is the delay of the train when it leaves the analysis area

This way to measure the punctuality/quality is e.g. used in the simulation tool SIMON.
7.3.1 Reasons for delay and consequences

Based on registrations made by Rail Net Denmark, the punctuality of the departure of the IC-trains from Fredericia station in April 1997 is estimated as follows:

![Graph showing punctuality for IC-trains at Fredericia Station (Fa) in April 1997](image)

Figure 7.5 Measurement of punctuality for IC trains at Fredericia Station (Fa) in April 1997 [4].

As appears from figure 7.5, at Frederia station in April 1997 it was not possible to meet the quality demand that at least 85 % of the IC-trains must be punctual (i.e. less than 5 minutes delayed), as $F(5 \text{ min}) = 76\%$.

However, it is characteristic of Fredericia station that the vast majority of the IC-trains have to change the composition of the train, and this procedure can have a big influence on the punctuality. Even though the timetable allows for prolonged stops at the station because of the coupling procedure, there may arise technical problems that will lead to delays as compared to the timetable. It appears from the below table that coupling problems is the second most important explanation for the delays of the IC-trains at Fredericia station in April 1997. It should however be noticed that the delay can only be explained in about 20% of the registered cases.
<table>
<thead>
<tr>
<th>Reason for delay</th>
<th>Category</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Another train</td>
<td>“Timetable”</td>
<td>72,6%</td>
</tr>
<tr>
<td>Reduced speed (LA, ATC or signal problems, left track)</td>
<td>Infrastructure/operation</td>
<td>5,8%</td>
</tr>
<tr>
<td>Shunting</td>
<td>Infrastructure/operation</td>
<td>1,2%</td>
</tr>
<tr>
<td>Coupling problems, formation etc.</td>
<td>Rolling stock</td>
<td>9,6%</td>
</tr>
<tr>
<td>Braking problems</td>
<td>Rolling stock</td>
<td>2,7%</td>
</tr>
<tr>
<td>Door problems</td>
<td>Rolling stock</td>
<td>2,3%</td>
</tr>
<tr>
<td>Control of or problems with software in the train</td>
<td>Rolling stock</td>
<td>1,5%</td>
</tr>
<tr>
<td>Other technical rolling stock problems (water/electricity/radio connection)</td>
<td>Rolling stock</td>
<td>2,3%</td>
</tr>
<tr>
<td>Waiting for train crew/catering</td>
<td>Crew</td>
<td>0,8%</td>
</tr>
<tr>
<td>Assistance to handicapped passengers, police</td>
<td>External</td>
<td>1,2%</td>
</tr>
</tbody>
</table>

Table 7.3 Reasons for delay of the IC trains at Fredericia Station in April 1997 [4].

Reasons for delay such as “Another train” suggest that the running time supplement is not big enough, that the buffer times are too small or the like. But as earlier described certain delays are allowed – also big ones which however must not occur too often. That a train is delayed by another train is therefore not necessarily due to an error in the timetable, but generally there will be a explanation that concerns an event that has taken place some time before the train arrives at the station, where the punctuality is observed. Direct errors/inappropriate dispositions in the timetable can however occur, but they will normally be corrected during the first weeks after a change of timetable.

If the imprecise explanation “Another train” is ignored, around ¼ of the operation troubles of the IC-trains is due to infrastructure conditions or operation problems which are not within the infrastructure owner’s responsibility. 2/3 of the identified reasons for delay can be referred to rolling stock problems, where coupling problems is typically the reason stated. Together with the cases where the train’s crew/catering is responsible for the delay, the operator is

---

8 As the month of April is just before a change of timetable, it is assumed that all direct errors or inappropriate dispositions in the timetable have been corrected at this point. “Another train” as reason for the delay should therefore be spread on the other categories “Infrastructure/operation”, “Rolling stock”, “Staff” and “External”. 

- 115 -
responsible for approx. 70% of the delays. The remaining 5% is due to external circumstances such as passenger conditions etc.

The distribution of explanations depends to a large degree on the line segment, the train class and time of year. For instance, the weather is not mentioned as a reason for delay of the IC trains at Fredericia station in April 1997, but in the autumn (with leaves on the tracks) and during the winter (with snow on the tracks) this explanation will normally occur more often. Nor is there any mention of many/“slow” passengers as reason for delay, which may be due to the line segment (+ the train category), as the dwell times for the IC trains at Fredericia station are timetabled very long due to the coupling procedure. Therefore there will normally not be problems with too little time for exchange of passengers to/from the IC trains at Fredericia station.

The reasons for delay that can be related to the infrastructure or the rolling stock will typically be regarded as technical errors. The duration of the technical errors plays a very important role with respect to the consequences of these errors. Regarding the rolling stock, the errors can be divided into [4]:

- **Transitory error**
  
  The error is corrected immediately, and it is not necessary to make actions, e.g. that other trains overtake the train with technical problems.

- **Permanent error, but the train can continue**
  
  Typically, it will be necessary to reduce the speed, e.g. because of reduced traction power.

- **Long-lasting error, where the train cannot continue**
  
  Until the train is taken out of the system or the error is corrected, the infrastructure occupied by the train cannot be used by other trains. Depending on the capacity of the line, this line obstruction will cause major or minor troubles in the operation.

Correspondingly, errors in the infrastructure can be grouped in the following way [4]:

- **Transitory error**
  
  The error is corrected immediately. It may e.g. be due to signal problems, where the driver must call on the radio to find out whether the train can continue.

- **Permanently reduced capacity**
  
  If the limitations of the infrastructure are prolonged, the traffic must be rerouted. It may be necessary to introduce
single track operation on a double track line, if one track is not passable.

- Total closure of the traffic
  
  If all the tracks on the line are blocked, the traffic can only be re-established when the error has been corrected.

All the above types of technical errors result in a limitation of the available capacity $K_{\text{disp}}$, as compared to the fundamental capacity $K_{\text{gr}}$ that can be obtained under normal conditions. In case of long-lasting limitations of the infrastructure or the rolling stock properties, these can be included in the system examined, after which it is possible to carry out a capacity analysis based on the new prerequisites. On the other hand, transitory technical errors should be examined in the original system, where the delay propagation from one train to a following train can be followed.

### 7.4 Delay propagation

When a travel time is extended beyond the optimum travel time, it may be because other trains in the system occupy the room that it needed to obtain an optimum travel time. This type of travel time extension will be defined as waiting time like the one known from the queuing theory. A travel time can easily be extended without being caused by a physical obstacle/limitation in the system. Therefore the concept *delay* that covers travel time extensions that cannot be expected beforehand (i.e. deviations from the timetable, cf. figure 7.3) is introduced. As appears from figure 7.6 there will be an overlap between the 2 concepts *waiting time* and *arrival*.

![Figure 7.6 Definition of waiting time and delay [3]](image-url)
The unexpected waiting time and the secondary delay will thus be 2 different ways to describe the same type of travel time extension. However, the secondary delay cannot be characterized as waiting time, since the trains in these situations in principle can depart from the station, but are held back to await passengers from another train. The initial delay is due to circumstances that happen before the train appears in the system. For instance, a train that does not arrive on time to the entrance station, e.g. because of problems at the storage sidings, will be inflicted an initial delay. It is not called waiting time, as it would then have to be directly related to the system itself. Typically, initial delays will be defined as primary delays, whereas secondary delays will be defined as secondary delays. When a primary delay propagates to another train, a secondary delay has arisen. In the following section it is shown how this delay propagation will develop in case of different infrastructure combinations (single/double track) and traffic composition (homogeneous/ inhomogeneous). This problem has previously been analysed in [5] and [8].

7.4.1 Delays on double track lines with homogeneous traffic composition

The simplest case is to describe a double track line, where there is only one-way traffic on each track is, and the trains are operated at the same speed (i.e. homogeneous traffic). Therefore, the general theory about delay propagation is described based on this example. One way to avoid big delay propagations is to introduce sufficiently big buffer times \( t_b \) between the trains. The size of the buffer times can be expressed in the following way:

\[
\text{Formula 7.4} \quad t_b = t_{h,tt} - t_{h,min}
\]

Where: 
- \( t_{h,tt} \) is the practical headway time that can be found by looking in the timetable
- \( t_{h,min} \) is the theoretical minimum headway time

Below it is assumed identical buffer times \( t_b \) between the individual trains.

Provided the primary delay \( p_1 \) is bigger than the buffer time \( t_b \), the arrival will spread to the following trains. These secondary delays \( p_{i+1} \) can be determined in the following way:

\[
\text{Formula 7.5} \quad p_{i+1} = p_i - t_b
\]
Where: The index to p indicates which train number the delay concerns. Index 1 is thus the train with the primary delay, whereas index 2, 3 etc. are secondary arrivals.

The delay will be reduced from train to train, provided that no new primary delays arise, as the buffer times will normally be positive. The last train influenced by a delay p₁ will thus have secondary delays p₁ that can be expressed as:

Formula 7.6 \[ p_{j+1} = p_1 - j \cdot t_b = \kappa \cdot t_b \quad \kappa \in \{0;1\} \]

Where: j is the number of trains receiving secondary delays.

Figure 7.7 thus shows the delay propagation to the 2 following trains that are inflicted secondary delays, after which the system is regenerated, i.e. j = 2.

Figure 7.7 Delay propagation on double track line with homogeneous traffic.

The sum of all the delays \( \Sigma p \) that one event can give rise to can be expressed as follows:

Formula 7.7 \[ \sum_{i=1}^{j+1} p_i = \sum_{i=1}^{j+1} \left( p_i - (i-1) \cdot t_b \right) = \frac{1}{2} \cdot (j+1) \cdot \left( 2p_1 + j \cdot t_b \right) \]

---

9 However, in rare cases the buffer times can be negative. They can e.g. be seen on local railways where the departure time rounded down to a whole minute.
The last rewriting can for instance be found looking in “Schaum Mathematical Handbook (19.1)”. By inserting formula 7.6 in formula 7.7 the total delay $\Sigma p$ can be expressed by means of the delay of the first and the last train, respectively.

Formula 7.8
$$\sum_{i=1}^{j+1} p_i = \frac{1}{2} \cdot (j+1) \cdot (p_1 + p_{j+1})$$

The above equation can be read as an expression that the total delay $\Sigma p$ is a product of the number of delayed trains $(j + 1)$ and their average delay $\frac{1}{2} (p_1 + p_{j+1})$. However, it is often convenient to express the total delay $\Sigma p$ only by the primary delay $p_1$ and the buffer time $t_b$. By means of a number of rewritings of formula 7.7, where the constant $\kappa$ from formula 7.6 can be used, it is possible to obtain an approximate expression for the total delay $\Sigma p$ which only depends on the primary delay $p_1$ and the buffer time $t_b$.

Formula 7.9
$$\sum_{i=1}^{j+1} p_i = \frac{1}{2} \cdot \left( p_1 - \kappa + 1 \right) \cdot \left( p_1 + \kappa \cdot t_b \right)$$

The last term in the above expression can at the most assume the value $\frac{1}{8} t_b$ (for $\kappa = \frac{1}{2}$). The last term can therefore with good approximation be ignored, as it is only relevant to determine the total delay $\Sigma p$, if the primary delay $p_1$ is considerably bigger than the buffer time $t_b$. A rough expression for the total delay $\Sigma p$ can therefore be described by:

Formula 7.10
$$\Sigma p \approx \frac{1}{2} \cdot p_1 \left( p_1 + 1 \right)$$

The above expression can also be obtained by inserting $p_{j+1} = 0$ in formula 7.8 and insert formula 7.6 in this expression in the following way:

$$\Sigma p = \frac{1}{2} \cdot (j+1) \cdot (p_1 + p_{j+1}) \quad \land \quad p_{j+1} = 0 \quad \land \quad p_{j+1} = p_1 - j \cdot t_b$$
The smaller the delay of the last train inflicted by a delay, the better the approximation in formula 7.10.

Formula 7.10 can be considered as a product of the primary delay \( p_1 \) and a propagation factor \( y(p_1) \), and it will thus be defined in the following way:

Formula 7.11
\[
y(p_1) = \frac{1}{2} \left( \frac{p_1}{t_b} + 1 \right)
\]

The propagation factor \( y(p_1) \) is thus a factor without a unit bigger than 1 giving a fixed expression of the extent of the consequences of a delay on for the entire system. A simple approximated expression for the total delay \( \Sigma p \) will therefore be:

Formula 7.12
\[
\Sigma p = p_1 \cdot y(p_1)
\]

When the total delay \( \Sigma p \) is known, the size of the total secondary delays \( \Sigma p_s \) can be determined as follows:

Formula 7.13
\[
\Sigma p_s = \Sigma p - p_1
\]

As it is often difficult to relate to the buffer time \( t_b \), in many correlations it is expedient to rewrite expressions that include the buffer time \( t_b \) with equation (4.3), thus the variable parameter will be the utilization ratio \( u \) and the theoretical minimum headway time \( t_{h,\text{min}} \) (= \( t_h \)).

Formula 7.11 will then appear as follows:
\[
y(p_1) = \frac{1}{2} \left( \frac{p_1}{t_b} + 1 \right) \land t_b = \left( \frac{1}{u} - 1 \right) \cdot t_h
\]
If the primary delay $p_1$ and the headway time $t_h$ is maintained, the delay propagation $y(u)$ as a function of the utilization ratio can be determined as shown in figure 7.8:

It appears from figure 7.8 that the propagation factor $y(u)$ will almost be doubled if the utilization ratio $u$ rises from 60% (the normally used maximum daily load) to 75% (the normally used maximum load for one hour), if the relation between the primary delay $p_1$ and the dimensioning headway time $t_h$ is 5.

In case of a dimensioning headway time $t_h = 2$ min and a primary delay $p_1 = 10$ min, the total delay $\Sigma p$ can be expressed in the following way by means of figure 7.8 or formula 7.14 and formula 7.12:

Normal time (utilization ratio $u = 60\%$): \[ \Sigma p = 10 \text{ min} \cdot \left( \frac{10 \text{ min}}{2 \cdot 2 \text{ min}} \cdot \frac{60\%}{1-60\%} + \frac{1}{2} \right) = 42\frac{1}{2} \text{ min} \]

Max time (utilization ratio $u = 75\%$): \[ \Sigma p = 10 \text{ min} \cdot \left( \frac{10 \text{ min}}{2 \cdot 2 \text{ min}} \cdot \frac{75\%}{1-75\%} + \frac{1}{2} \right) = 80 \text{ min} \]
It should be noticed that in the above calculation it is assumed that it is not possible for the trains going behind to overtake the train inflicted by the primary delay $p_1 = 10$ min.

With utilization ratios above 80%, it appears from figure 7.8 that the propagation factor increases dramatically. Therefore it seems absolutely reasonable that the upper limit of the utilization ratio is normally set at 75% during the peak hour, as major traffic load A will give rise to a considerable risk of very big and long-lasting secondary delays.

### 7.4.2 Delays on double track lines with inhomogeneous traffic composition

When the operation speed of all the trains on a double track line is not the same, the expressions from section 7.4.1 Delays on double track lines with homogeneous traffic composition cannot be used without further considerations. As appears from figure 7.9 the timetabled headway time $t_{h,tt}$ will depend on which trains are following each other. Furthermore, the timetabled headway time will depend on where on the line the observation is carried out.

![Figure 7.9 Delay propagation on double track line with mixed traffic.](image)

If the conditions at exit from an overtaking, entrance or junction station, respectively, are taking into the consideration, the headway will be of the order indicated in in figure 7.9 in case of the headway fast train (e.g. an IC-train) followed by a slow train (e.g. a regional train). However, in case of a reversed headway the headway time can be determined in the following way:

- $t_{h,tt,IC-Re}$
- $t_{h,tt,Re-IC}$
- $t_{h,tt,IC-Re}$
- $t_{h,tt,Re-IC}$
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Formula 7.15 $t_{h,tt,Re-IC} = t_{h,tt} + \Delta t_h$

Where: $\Delta t_t$ is the headway time that compensates for the speed difference between two trains

Also the delay propagation depends on the headway. In case of a headway where train $i$ is a fast train followed by a slow train, the delay of train $i+1$ is determined in the following way:

Formula 7.16 $p_{i+1} = p_i - t_b - \Delta t_h$

In case of other headway times $\Delta t_t = 0$

It appears from figure 7.9 that $p_1 - p_2 < p_2 - p_3$. This is in compliance with formula 7.16, as a inequality that compares the headway slow train followed by fast train with the reversed headway. It is extremely difficult to set up a general expression for the total delay $\Sigma p$ for all the trains when the traffic is inhomogeneous. It will depend on which train in the headway that is affected by the primary delay, and how the following headways affected by this delay are made up. Under the assumption that there is the same probability for all the trains to be affected by the primary delay, formula 7.8 expresses the average total delay $[8]$. If the average total delay has to be expressed as a function of the primary delay $p_1$ and the buffer time $t_b$, it is necessary to determine an expression of the average buffer time $\bar{t}_b$, as the buffer time $t_b$ in formula 7.4 only applies to homogeneous traffic. In case of a timetable with a cycle consisting of a pool with $N_{IC}$ fast trains and an equivalent pool with $N_{Re}$ slow trains, the average buffer time $\bar{t}_b$ can be expressed in the following way:

Formula 7.17 $\bar{t}_b = \frac{(N_{IC} + N_{Re} - 1) \cdot (t_{h,tt} - t_{h,min}) + t_{h,tt} + \Delta t_h - t_{h,min}}{N_{IC} + N_{Re}} = t_{h,tt} + \frac{\Delta t_h}{N_{IC} + N_{Re}} - t_{h,min}$

If $t_b$ is replaced by $\bar{t}_b$ in formula 7.7, formula 7.9, formula 7.10 and formula 7.11, the total delay $\Sigma p$ can be determined, provided that a total number of cycles are affected by the secondary delay, i.e. if $j/(N_{IC} + N_{Re}) \in Z$. Otherwise, the equations will only be a rough estimate. As you will typically need to determine the delay propagation on average instead of at specific circumstances, the prerequisites used in this section will normally not influence the applicability of the equation negatively.
Apparently it seems that the delay propagation is smaller in case of an inhomogeneous traffic composition as compared to a homogeneous traffic composition, since $t_b$, cf. formula 7.17, is bigger than $t_b$, cf. formula 7.4. However, this is not the case, as an inhomogeneous traffic composition will mean less trains at the same utilization ratio $u$ (and average buffer time $\bar{t}_b$) as homogeneous traffic. If the delay propagation in 2 different timetables is compared, it should be done based on the same traffic intensity $q$.

7.4.3 Delays on single track lines

On single track lines, it is difficult to describe the delay propagation in general terms, as the primary delay most likely will affect both directions of travel. In case of major operation troubles, it is not unusual to change the operation strategy, so that there will be deviations from the mutual order in which the trains are timetabled to operate the line. It will therefore be very complicated to make a general evaluation of the consequences of a primary delay. However, Potthoff has set up the below expression that explains the total delay $\Sigma p$ caused by a primary delay $p_1$ on a single track line consisting of similar line sections that are operated by trains with the same operation speed, cf. figure 7.10.

Formula 7.18 $\Sigma p = \frac{1}{2} \cdot p_1 \cdot \left( \frac{p_1}{t_b} + 1 \right) + \frac{1}{2} \cdot (a - 1) \cdot n \cdot \left( p_1 - n \cdot t_b \right)$ [5]

Where: $n$ is the number of trains driving in the same direction just after each other (i.e. in a convoy)

However, to use the equation it is a prerequisite that it is a train going in the primary direction (cf. figure 7.10 Station A → Station E) that is inflicted the primary delay $p_1$, and that trains going in the primary direction do not have to await crossings with trains going in the opposite direction as shown in figure 7.10. In case of a line section with single track (i.e. $a = 1$), formula 7.18 becomes identical to formula 7.10 meaning that the delay propagation in this case will be the same for single track lines and for double track lines.

If the single track line consists of several line sections (i.e. $a > 1$), the trains going in the secondary direction will be delayed equivalent to the 2nd term of formula 7.18. This term can be explained by the fact that a train in the secondary direction is on average delayed the size $(p_1 - t_b)$ in each of the $(a - 1)$ extra block sections. If $p_1$ is set outside the brackets for both terms in formula 7.18, the total delay $\Sigma p$, cf.

\[10\] A line section is defined as the line between 2 crossing stations.
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Formula 7.12, can be conceived as a product of the primary delay $p_1$ and the propagation factor $y(p_1)$ that can be determined in the following way:

\[
y(p_1) = \frac{\sqrt[2]{p_1}}{1 + n \cdot (a - 1) \cdot \left(1 - \frac{n \cdot t_b}{p_1}\right)}
\]

As earlier described it only makes sense to calculate the secondary delays, if the primary delay $p_1$ is bigger than the buffer time $t_b$. In formula 7.18 and formula 7.19 there should furthermore be a whole number of cycles affected by the initial delay meaning that $p_1 > n \cdot t_b$

When comparing formula 7.11 and formula 7.19 it is therefore no surprise that the delay propagation on single track lines is at least as big as the one on double track lines (but normally bigger)\(^{11}\).

In figure 7.10 is shown how a primary delay $p_1$ will spread to many of the following trains. However, all trains will be less delayed than the primary delay $p_1$, as the operation strategy (i.e. the mutual order of the trains) is not changed due to the operation troubles.

\(^{11}\) In situations where the equations are valid
As appears from the equations in this section, they contain several prerequisites and limitations. The general formulas can therefore only be used in very idealized situations.

If the exact location and time of the primary delay is known, it is however possible to determine the delay propagation without using the general formulas. If the rules of dispatching have been established, it is possible to create the graphic timetable and use it for determining the individual delays (cf. figure 7.10). However, if you wish to examine the consequences of a random primary delay, it is convenient to use a simulation model to describe the delay propagation, if the general formulas cannot be used.

Figure 7.10 Delay propagation on single track line with homogeneous traffic \((a = 4)\) and \((n = 2)\).
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Chapter 8: Network Effects

The aim of many railway investment projects in Europe is to remove bottlenecks in the infrastructure, thereby making it possible to reduce travel times and increase the number of trains. The size of the benefits depends on the actual project involved. In any case, it is an important planning task to determine the future timetable. This makes it possible to determine the benefits for travellers in a standard cost-benefit analysis (CBA). The scope of this chapter is to discuss the influence of the size of the analysis area on the calculated travel times.

Now, once a project is completed it is fairly straightforward to recognize that the project will also influence the future timetable outside the project area, thereby creating benefits outside the local area of the project. This is denoted "Network effects" in this chapter. It is not possible to claim this as a general effect.

On a rural branch-line, where local trains connect to the national InterCity system, it is unlikely that any project will influence the national timetable. The benefits on the rural branch-line will be purely local, while removing a major bottleneck on the main-line network is likely to create changes on many connecting lines, and maybe improve the timetable across the national network. In this case, a local analysis is not sufficient to capture all benefits.

8.1 Network effects in the Danish context

An example to illustrate the network effects is the Danish railway line between Aalborg and Frederikshavn, cf. figure 8.1. It is a single track line with a one-hour service. The travel time in one direction is 63 minutes and 66 minutes in the other direction. The speed on the line is increased from 120 km/h to 180 km/h.

This project can be evaluated locally. However, the traffic in the northern part of Denmark is not timetabled independently of the remaining network. The trains are part of the nationwide IC system and are therefore adapted to the arrival and departure times of the IC-trains at Aalborg (as well as the crossing possibilities in the northern part of Denmark).

If the crossing in the candidate timetable for the upgrading project is moved to obtain benefits locally, e.g. 10 minutes for one of the directions, it would result in nationwide changes. It is due to the fact that most regional trains have connection to and from IC-trains. A change in the northern part of Denmark will therefore influence the regional trains Copenhagen – Nykøbing F (in the southern part of Denmark) because of the connection at Ringsted cf. figure 8.1. This change may very well result in time benefits (or losses) at other lines of the network.
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Figure 8.1 The Danish railway infrastructure.

Since network effects can influence the entire railway network, the extent of which are impossible to examine, it is necessary to delimit an area of analysis. Therefore, it is interesting to examine the importance of these network effects.
8.2 Methodical overview

To identify the network effects of the above mentioned increased speed project, a nationwide candidate timetable for one standard hour must be worked out. However, the nationwide timetable in Denmark depends on the train services to/from Germany and Sweden. To evaluate all the network effects it is therefore not enough to create a nationwide candidate timetable. It is necessary to include the trains to/from Germany and Sweden and thereby also the nationwide timetables of Germany and Sweden and so forth.

Network effects can be illustrated by queuing time. Queuing time is the difference in running time when comparing a single train on a line with a situation with many trains on the line. Queuing time on railway lines occurs when the traffic intensity is close to the capacity level due to e.g. mixed operation (slow and fast trains). When close to the capacity level, the operation speeds of fast trains must/will adapt to the slower trains cf. figure 8.2. This will increase the travel time for the trains that under free conditions could run at higher speeds [8].

![Figure 8.2 Extended running time (queuing time) due to other trains on the line](image)

To calculate the queuing time the Danish developed SCAN model (Strategic Capacity Analysis of Network) can be used (a similar function is found in the German tool UX-SIMU). SCAN is a computer tool for calculation of capacity in a railway network. In SCAN capacity is measured as average queuing time in a sample of candidate timetables for a given infrastructure alternative. The tool can be used

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1 Cf. Chapter 3 “The Capacity concept” for details about queuing time
in the strategic planning process where the exact infrastructure and timetable are not determined. Therefore the system is based on a structure where it is only necessary to know the plan of operation (i.e. the number of trains within each category), the infrastructure in a simple way and the main dynamics of the rolling stock [4].

8.2.1 The general problem of Network effects

One of the problems that have to be handled when calculating the consequences of a project is the size of the analysis area. When the analysis area is large, the risk of network effects is high. This is due to the fact that when a large analysis area is examined it is because of bigger changes in the infrastructure and/or timetables. Major changes in the infrastructure and/or timetables may influence many trains in the analysis area, and these trains may influence other trains outside the analysis area.

However, even smaller analysis areas may generate network effects. This is due to the way of planning the timetable in Denmark and many other countries. All train services can be placed in a hierarchy, cf. figure 8.3, where the train services placed in the top of the hierarchy is planned and timetabled before trains further down in the hierarchy.

```
  International
    ↓
  InterCity Express
    ↓
  InterCity
    ↓
  Inter Regional
    ↓
  Regional
    ↓
  Local
```

**Figure 8.3 The hierarchy of the train service.**

Even small changes in the timetable of a train in the upper level of the hierarchy may influence other trains further down in the hierarchy, because these trains are planned according to the train high up in the hierarchy. Since trains high up in the hierarchy often travel long distances, the changes for other train services can occur far away from the analysis area.

These changes in the railway network can occur far away from the analysis area is due to the network effects. Therefore, it is important to examine whether there are network effects. Nonetheless, the effects
are normally only studied locally. It can be due to lack of resources, or because the network effects are uncertain (or insignificant), or because you only wish to evaluate the project locally, isolated from the remaining network.

8.2.2 Network effects between Copenhagen and Ringsted

An example of a local study is found in the Copenhagen-Ringsted study where it was examined whether to extend the existing railway line or build an extra railway line in another layout. The capacity analysis for each of the alternatives only covers the actual line Copenhagen-Ringsted. For instance, the capacity conditions at Copenhagen Central Station have not been included [7]. The analysis itself consists of calculated queuing time for basis and 3 main alternatives as well as a variant of the new construction solution see figure 8.4. The analysis can be used to rank the alternatives examined, but it does not provide information about the conditions of the project alternative, when the capacity conditions on the adjacent lines are included.

![Figure 8.4 Queuing time analysis of the Copenhagen-Ringsted alternatives (based on [7])](image)

However, when an infrastructure project is carried out, the traffic on the line or station in question is of course not planned independently of the remaining network. As the yearly timetable is planned at the network level, the timetable for the project area will also be influenced by this. The extent of this influence depends entirely on local conditions. In case of major improvements of the infrastructure on the main lines, it influences the timetable for the entire country (due to connections between long distance trains and regional trains).

8.2.3 Methodical results

The influence is bilateral, partly because a candidate timetable covering the project area may have derived effects at the network
level, but the opposite can also be the case. If a candidate timetable is only worked out locally, it is not examined whether this timetable has conflicts outside the area. At worst, the candidate timetable in question gives rise to conflicts on the adjacent lines in the immediate area. Therefore, it cannot be implemented in real life because of capacity conditions on the remaining network and it thus overestimates the benefits of the project.

The problem will be most significant, if the project is adjacent to lines with dense traffic/high capacity utilization. When preparing the timetables you normally start with the lines forming bottlenecks within the network, and if the project is adjacent to such a bottleneck, the timetable for this line will determine the timetable for the project area.

A way to recognize the network effects is to work out a candidate timetable for a larger part of the network than the project area. The size of the network to be included will depend on the project (and the analysis level). This method is used in the Netherlands where the network is more complex than in Denmark. Here a candidate timetable for one standard hour is worked out for the entire network. This is done by the Dutch tool DONS [6]. The timetable is then used to evaluate a project, so that it reflects both local effects and network effects.

It is a methodical problem that network effects cannot be generalized. Analyses of queuing time can be used for an initial evaluation of the network effects. By first carrying out an analysis of the queuing time locally and then an analysis including the adjacent lines the difference can indicate the problem. If the level of queuing time increases (queuing time per train-km) it is necessary to look at the network effects, but if the level decreases they can (probably) be ignored. This method is tested in the following section.

8.3 Network effects illustrated by queuing time

In this section the size of the network effects is examined for the Copenhagen – Ringsted example by means of the queuing time method. After the public hearing in 1998, a new examination of the project was initiated in 1999, including the development of a new traffic model. For the alternatives to be examined, a set of candidate timetables were prepared.

Three of these alternatives have been selected and used in this section. With regard to the infrastructure, the alternatives are as follows:
The candidate timetables comprise the railway network in Zealand (except from the railway line between Elsinore and Copenhagen Airport) and Funen and include the line Copenhagen Central Station-Fredericia (just west of Funen), cf. figure 8.1 and figure 8.5.

With respect to this chapter these timetables have been reduced to plans of operations by eliminating all arrival and departure times, and the queuing time is then simulated in the tool UX-SIMU. In this model,
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Queuing time is calculated as the average of all “timetables”. The result is minutes of queuing time per train-km.

Two studies of network effects are carried out:

1. The importance of the size of the analysis area
2. The importance of including connections

In the first calculation the analysis area is reduced to include only the project area locally. The queuing time is calculated for the same trains, but only on this line, so that it is only the capacity conditions on the approximately 60 km long line that influence the queuing time.

In the other study the original analysis area is maintained, but now 2 connections are included in the calculation of the queuing time. A demand for connection will reduce the number of possible candidate timetables and give a more realistic calculation. Traditional analyses of queuing time do not cover this, and it is an example of an obvious calculation simplification. It is not necessarily clear which transfers there will be between the trains in a future scenario, it is however taken for granted that there will be connections somewhere in the network. The need for resources is of course smaller if the connections are ignored, and for capacity analyses, where several projects have to be ranked, it is probably acceptable.

8.4 The size of the analysis area

The importance of the analysis area has been illustrated by calculating the queuing time for the whole of East Denmark as opposed to only calculating the queuing time for the line that is extended, i.e. the Copenhagen-Ringsted line. In East Denmark the capacity conditions Copenhagen-Fredericia (approx. 200 km) are included as well as the adjacent lines in Zealand where also single track lines are included.

The result is made up as average queuing time per train-km, as the 3 main alternatives do not include the same number of train departures. The result appears from figure 8.6. The average queuing time for a train that covers a distance of 100 km is for instance 4.2 minutes in the basis scenario.
In both Basis and Full extension (New line) scenarios it is seen that the queuing time drops considerably, if it is only calculated locally as opposed to a bigger part of the network. An isolated local examination will therefore underestimate the queuing time when the project is seen in connection with the remaining network. In New line, with 4 tracks to Ringsted, it reflects that the bottlenecks are now found on lines further away than Copenhagen-Ringsted. In the extending line scenario the amount of queuing time is more or less the same, so in this case there is a balance between traffic and infrastructure.

The figures of queuing time indicate that the capacity conditions are underestimated when you only look at the effects locally. It can also be interpreted in connection with the timetables and the travel times indicated in these. If the candidate timetable only comprises the line locally, then the possible travel times are overestimated. When the remaining part of the network is included, it results in an increase of the queuing time. In other words, travel times, as a consequence of the project, will be longer than shown by a local calculation of the travel times Copenhagen-Ringsted. Once completed, locally the timetable will depend on the remaining network.

Furthermore, when comparing the queuing time for the entire network in the 3 alternatives it is seen that both extension alternatives have relatively higher traffic intensity than basis. It can be interpreted in such a way that the extensions are trafficked too heavily as compared to basis; i.e. the regularity will be relatively worse than at basis (i.e. the socio-economic benefits of the projects are overestimated).
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8.5 The importance of connections

In many (Danish) cases analyses of queuing time do not take transfers into consideration. To examine the importance of this simplification, it has been examined how much the above calculated queuing time for the whole of East Denmark is influenced, if a number of connections are taken into consideration.

The calculation has been made with 2 commonly occurring connections: At Ringsted between an IC-train and a regional train Copenhagen-Nykøbing F and at Roskilde between an IC-train and a regional train Copenhagen-Holbæk (cf. figure 8.1). The calculation has only been made for the extension solution, the new line scenario.

![Graph showing queuing time](image)

Figure 8.7 Queuing time Copenhagen-Ringsted with connections (based on[1] and [2]).

It appears from figure 8.7 that the amount of queuing time increases a little in the New line scenario. However, as only very few of the trains examined are subject to connection demands, it requires further studies to make a general statement.

On the other hand it can be concluded that network effects can have a big influence on the locally obtained travel times, and thereby on the calculated socio-economic benefit. As a principal rule, candidate timetables are worked out for an area that is larger than the project itself to include the effect of the capacity conditions on the adjacent lines. As a supplement to this a sensitivity analysis can be carried out by means of a local timetable. In a number of cases the local timetable will give shorter travel times and a better socio-economic profit and illustrate the effect of bottlenecks in the adjacent network.
8.6 Summary

This chapter has presented network effects and a method to evaluate the network effects by examining the queuing time in the timetable. It has furthermore been shown that network effects are likely to occur in a railway system.

Changes in the timetable at one place of the railway network can lead to changes in the timetable for train services and/or railway lines far away. These changes on other train services and/or railway lines far away are due to network effects. Network effects are most likely to occur if changes in e.g. the infrastructure in a large analysis area are examined or if a train high up in the hierarchy of train services is influenced by the analysis.

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