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TEN YEARS OF QUALITY CONTROL IN ROAD CONSTRUCTION IN THE NETHERLANDS

by

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The views in this article are the authors' own.

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1 Introduction

Historical background

After ten years experience – involving more than 300 projects – of the system of quality control introduced in 1968 and associated with reduced-payment clauses, the State Road Laboratory has more recently developed an almost entirely statistical method.
The system which still exists at present is described in a detailed publication by Van de Fliert and Brouwers in the July-August 1968 issue of 'Wegen'[1].
That article also outlines the historical background in this area. Developments since 1960 can be briefly summarized as follows.

In the years 1960 to 1965 the first steps were taken towards the introduction of a general method of quality control for the carriageway pavements laid in a number of large highway projects; this ultimately led to an appropriate system which was accepted both by the public authorities and by the contractors' organizations.
On the basis of the large volume of data amassed over the years, contract specifications involving reduced-payment clauses were experimentally introduced for some projects in 1966. The clauses and specifications were completed in the next two years. The final version was then adopted in 1968.

At the time these developments were encouraged by a lack of sufficiently qualified, supervisory staff in the directorate and, above all, by the increasing mechanization and automation of construction work together with the growth in the scale of most projects.
Both the public authorities and the contractors gradually reached the conclusion that it was no longer acceptable to apply unilateral quality control under the sole responsibility of the directorate and normally based on analysis of a relatively small number of samples generally taken by a selective method. This method was found to be outdated and in need of fundamental change.

The clauses incorporated in the contract specifications since 1968 were adjusted in points of detail in 1972 and 1975 following a review of the general criteria and test methods for materials, mixes and surfaces. They have thus undergone no basic change since 1968. However, in recent years an alternative system has been developed, based on statistical principles and on the extensive experience of sample-taking and testing acquired over the past ten years.

Consultation between the directorate and contractors

Before the definitive system of quality control with reduced-payment clauses was introduced in 1968, consultation took place between the Public Works Department (State Road Laboratory) and a committee of expert representatives of the contractors. These consultations were intensified in the seventies when the 'Specifications for construction and control of road pavements' (VUCW) and the detailed 'Recommendations for production control in road building' (ABCW) were compiled; these texts were published in 1975. The VUCW were revised in 1977-78, again in close consultation between the public authorities and contractors' organizations, and published as the VUCW 1978.

A favourable situation obtains in the Netherlands in that the State Road Laboratory (RWL) is a central, public body with responsibility, in the road building sector, for compiling the criteria for materials and working methods, for the structural design of highways and the composition of the mixes used, for quality control specifications and for the performance of quality control in the case of state highways. This central role has led to a high degree of uniformity in the specifications applicable in the Netherlands and also to effective consultation with the representative committee of the contractors' organizations.
The system of quality control which has been applied in the past ten years has been accepted in broad outline by the Netherlands contractors' organization and recognized as reasonable, equitable and effective in maintaining the desired quality level.

2 Non-statistical system (1968-1978)

General principles

The system is based on a clear distinction between daily production control under the responsibility of the contractor on the one hand, and limited acceptance control by the directorate on completion of the works on the other. To ensure good mix characteristics, the contract specification stipulates that the contractor must effect preliminary studies to determine the thickness (and the requisite cement content) of the sand-cement roadbase, and also the composition and Marshall stability of the asphalt mixes (mix design).
The results of these preliminary studies are compared with the results of similar investigations conducted by RWL.
In this way specific agreements are reached between the directorate and the contractor on the design and characteristics of the mixes before the actual work begins.
In principle the method of implementation is thus the main determining factor as regards the standard of quality of the pavement layers.

Production control by the contractor

The specification stipulates that the contractor must effect thorough daily controls of the composition and characteristics of the mixes; these controls are governed by the same provisions as the acceptance control.
The contractor must therefore have a well-equipped site laboratory at his disposal with qualified personnel.
For major works in particular, the contractors regularly use statistical methods to control production quality; control cards are utilized for this purpose. On the basis of the data obtained in this way it is possible to control the procedures for mixing, compacting and processing the road building materials.

Acceptance control by the directorate

Quality control in respect of road building in the Netherlands relates in the first instance to the following main characteristics:
- layer thickness;
- bitumen content of the various types of asphaltic concrete;

– void content of asphaltic concrete and degree of compaction of sand asphalt;
– compressive strength of sand cement.

In the past 15 years no cement concrete pavements have been laid on trunk roads in the Netherlands road network. Should pavements of this kind be applied in the future a system of quality control similar to that now used for asphalt pavements would have to be developed for concrete road surfaces.

Under the system in use since 1968 one sample is taken per 2000 m^2 asphalt pavement; this sample consists of two cores with a diameter of about 100 mm drilled out of the completed pavement. If analysis of the characteristics listed above shows the quality

Table 1 Table of penalties*

layer thickness	*penalties per 2000 m^2 – in guilders*		
shortfall on thickness	roadbase: 0.15 m sand cement or 0.12 m sand asphalt	basecourses of 0.06 m bitumen-bound gravel (per course)	basecourse or wearing course of 0.04 m asphaltic concrete
1– 5 mm	–	–	–
6–10 mm	–	–	2000
11–15 mm	–	2000	4000
16–20 mm	1000	4000	6000
21–25 mm	2000	6000	8000
etc.	etc.	etc.	etc.

compressive strength: sand cement		*bitumen content: asphaltic concrete*		*voids: asphaltic concrete*	
strength too low**	penalty per 2000 m^2 in guilders	bitumen content too low	penalty per 2000 m^2 in guilders	voids too high	penalty per 2000 m^2 in guilders
0.1–0.5MN/m^2	1000	0.1–0.2%	–	1%	–
0.6–1.0MN/m^2	2000	0.3–0.4%	1000	2%	1000
1.1–1.5MN/m^2	3000	0.5–0.6%	2000	3%	2000
1.6–2.0MN/m^2	4000	0.7–0.8%	3000	4%	3000
2.1–2.5MN/m^2	5000	0.8–0.9%	4000	5%	4000
etc.	etc.	etc.	etc.	etc.	etc.

* The amounts quoted in this table were applicable for the years 1974-1977. At the beginning of 1978 most of the penalties were increased by 50%.

** Strength shortfall with reference to the criterion of at least 2.0 MN/m^2 applicable for the period 1972-1977. At the beginning of 1978 this requirement was reduced to 1.5 MN/m^2.

Figure 1. Cores, drilled out of the pavement, intended for testing in relation to acceptance control of pavements.

to be inadequate, financial penalties are imposed. These penalties are determined by the mean result for the two core samples. As a function of the gravity of the deviation from the required values, the flat-rate penalty ranges in practice from 1,000 to 10,000 guilders per sample of 2000 m^2 and per characteristic of a given layer, e.g. compressive strength of the sand cement roadbase or voids of the asphaltic concrete surfacing. (See Table 1.)
The system employed up to now is clearly not a genuine statistical system in the strict sense of the term since it is based on determination of the quality of individual samples and on penalties fixed in the light of the analysis results. In practice, however the number of samples is normally so large (e.g. 50 for a controlled surface area of 100,000 m^2) that we have in effect a non-selective random sample capable of giving sufficient information on the quality of the work in its entirety. The results of analysis and their statistical processing lead to the same conclusion: in most cases the number of insatisfactory samples–i.e. in excess of the penalty limit–expressed as a percentage of the total number of samples is roughly the same as the number calculated theoretically from the mean and the standard deviation. In addition, the system implies that 2% of the total number of samples may show results which fail to meet the specified criteria without giving rise to penalties. On the other hand higher percentage deviations do automatically result in penalties.

In this context it is important to note that the conclusion as to whether the work as a whole is 'good' or 'bad' does not depend on just one characteristic of one component part of the road construction; each project is assessed in the light of the test results for at least 3 or 4 characteristics:
layer thickness, strength of the sand cement (where this is used), and the density and bitumen content of asphalt mixes. In addition the tests always relate to at least 3 or 4 different layers: sand cement (15-40 cm) or sand asphalt (10-12), bitumen -bound gravel (12-24 cm), open-textured asphaltic concrete (4-8 cm) and dense asphaltic concrete (4 cm) – see Table 2.

To sum up, the overall system is thus a combination of in general some 10 different sub-systems of quality control.
In a sense the 'risks' are thus spread over the entire construction. For example, if a penalty of 1% must be imposed because of insufficient strength of the sand cement while no other penalties are charged in respect of the other characteristics and layers, the total penalty will be limited to about 0.2% of the overall value of the construction project. If on the other hand the overall penalty for a particular project is high, it may safely be concluded that the quality standard of the project as a whole is low.

Acceptance control definitely does not make routine site supervision by the directorate superfluous – quite the contrary.
Acceptance control (by RWL) on completion of the works relates only to certain spe-

cific (though essential) aspects of the construction work which are in any case only tested on a random sample basis. Careful supervision by the local directorate (which also supervises and takes regular note of the results of the contractor's quality control) can reveal, or better still prevent, extreme – and also incidental – faults. If the results are unsatisfactory the contractor himself must take direct action and the directorate will also ask for shortcomings to be remedied.

Results

The principal results obtained in quality control of more than 300 projects since 1968 are summarized in Tables 2 and 3.
These projects involved areas of at least 50,000 m^2, generally between 100,000 and

Table 2 Test results

property	material	overall mean value $\bar{\bar{x}}$ $(\sim\mu)$	mean standard deviation $\bar{s}(\sim\sigma)$	specification or penalty limit R	ξ	δ	quality number Q_{calc}	quality number Q
compressive strength MN/m^2	sand cement	6.0	2.3	2.0	1,74	4,0	1,36	1,40
relative density (Marshalltest), %	sand asphalt	98.0	2.0	94.5	1,75	4,0	1,36	1,40
voids, % (V/V)	bitumen-bound gravel	5.9	1.8	9.5	2,00	2,3	1,60	
	open-textured asphaltic concrete	4.7	1.9	8.5	2,00	2,3	1,60	1,60
	dense asphaltic concrete	3.7	1.65	7.0	2,00	2,3	1,60	
bitumen content, % (m/m)	bitumen-bound gravel	5.0	0.32	5.0 ± 0.75	2,34	1,0	1,65	
	open-textured asphaltic concrete	5.5	0.31	5.5 ± 0.75	2,34	1,0	1,65	1,60
	dense asphaltic concrete	6.5	0.29	$6.5+0.75$ $6.5-0.65$	2,24	1,25	1,57	

property	material	overall mean value $\bar{\bar{x}}$ ($\sim\mu$)	mean standard deviation $\bar{s}$ ($\sim\sigma$)	specification or penalty limit R	ξ	δ	quality number Q_{calc}	quality number Q
layer thickness, mm	sand cement	150	17	120				1,40
	sand asphalt	120 (130)	18	100				1,40
	open-textured asphaltic concrete	40 (43)	8	30				1,40
	dense asphaltic concrete	40 (43)	6	33				1,40
	total asphaltic concrete (120 mm bitumen-bound gravel, 40 mm open-textured asphaltic concrete, 40 mm dense asphaltic concrete)	200 (216)	20	180				1,40

The layer thicknesses indicated in the 3rd column are nominal values, specified as minimum thicknesses. In the case of asphalt mixes, prescribed quantities must be processed: 20 kg/m^2 and 25 kg/m^2 per 10 mm nominal thickness for sand asphalt and all types of asphaltic concrete respectively. The processed quantities are measured by weighbridge and charged up to this maximum.
Since the normal mean degrees of compaction of sand asphalt and asphaltic concrete are approximately 1850 kg/m^3 and 2300 kg/m^3 respectively, an extra 'safety margin' of about 8% is included for each asphalt layer in order to ensure the presence of minimum (nominal) thicknesses throughout; the mean effective thicknesses are shown in brackets in the table.
The bitumen content values in the 3rd column are also specified as prescribed nominal values.

200,000 m^2 and sometimes even more. The majority of projects thus had a length of 10-20 km with an average carriageway width of about 10 m.
The overall average values for $\bar{\bar{x}}$ (μ) and the mean standard deviations $\bar{s}$ (σ) shown in the second and third columns of Table 2 were calculated as follows.
The random sample average ($\bar{x}$) and the (estimated) standard deviation (s) were determined for each project and each characteristic.
Using the values for $\bar{x}$ and s, the mean value of the random sample averages ($\bar{\bar{x}}$) and the mean standard deviation ($\bar{s}$) were then calculated for each characteristic.

Where the construction work is performed to a proper standard and sufficient care is taken over quality control, the completed works meet the specified criteria and there are few, if any, penalties.

Table 3 Level of penalties with reference to number of controlled projects (1968-1975)

penalties not higher than guilders/t asphalt	number of projects % (cumulative)
0	9
0.1	35
0.2	54
0.3	67
0.4	77
0.5	80
1.0	91
2.0	96
3.0	98
4.0	99
5.0	100

Current average cost per ton of asphalt in the Netherlands: approx. 50 guilders (approx. $ 25).

3 Statistical system

General

The system of quality control for asphalt pavements used up to now is, as we have seen, based largely on determination of the quality of individual cylindrical samples extracted by drilling from the completed pavement; the penalty is always related exclusively to the results of the tests carried out on these cylindrical samples.
This essentially traditional method of quality control has certain drawbacks.
Information which has become available in the past ten years or so (as in other technical sectors) shows that these drawbacks can be overcome by applying statistical methods.
In a statistical quality control system, interpretation of test results on the basis of mean values and standard deviations replaces the analysis of individual samples.
The possibility of switching over to a fully statistical system was already discussed in the article by Van de Fliert and Brouwers (1968) referred to above; the principle referred to by them of relating the penalty level to the statistical excess percentage can also be considered as the basis of the new system which was described in detail in an earlier publication [8].

Testing system

Rational quality control must meet the following requirements:
a. The testing system must be such that acceptable works do not normally incur penalties while a penalty of any importance is only imposed on them in exceptional cases (producer's risk). On the other hand the testing system must be designed in such a way that 'bad' works do normally incur substantial penalties; this is very important, if only indirectly, as a preventive measure to encourage good quality control.
b. Because the quality of the works is determined on the basis of the results obtained by means of a non-selective random sample of limited size, 'chance' will have a relatively large influence on the results.
In others words the magnitude of the random sample average ($\bar{x}$), and that of the standard deviation (s), are influenced by chance. The testing system must allow for such accidental deviations.

The term 'testing against variables' is used when the acceptance or rejection of a batch of products or the imposition of a penalty on the contract sum for a particular project depend on variables such as the mean ($\bar{x}$) and standard deviation (s) of observations on a random sample taken from the batch or on core samples drilled from an asphalt pavement.

The aim is thus to keep the mean and standard deviation under control. A system of this kind has now been chosen for acceptace control of asphalt pavements and sand cement roadbases. In addition to the two parameters $\bar{x}$ and s, in testing these pavements and roadbases against measurable characteristics such as the voids content of asphaltic concrete and the compressive strength of sand cement, two further values must be known:

- a limit value or penalty limit R below or above which a particular measurable characteristic must be defined as functionally 'bad';
- the uncertainty percentage which is still just acceptable; a value frequently encountered in industrial practice is 5%.

As mentioned above, the testing system must be designed in such a way that penalties may only occur in exceptional cases for acceptable works; a figure of 0,05 is often taken for this 'producer's risk' – in other words a likelihood of approval (or, expressed differently, no imposed penalty) of 95%.

The following system is now used: the physical characteristic to be investigated is measured on a non-selective random sample of the prescribed size n. From the n results obtained both the mean

$$\bar{x} = \frac{\sum_{i=1}^{n} x_i}{n},$$

and standard deviation

$$s = \sqrt{\frac{\sum_{i=1}^{n} (\bar{x} - x_i)^2}{n - 1}}$$

are determined.

With the aid of these results, the testing parameter

$$\frac{R_{\max} - \bar{x}}{s} \text{ or } \frac{|R_{\min} - \bar{x}|}{s}$$

is calculated, depending on whether the penalty limit R is a maximum or minimum value. In one instance, namely determination of the bitumen content, where there is both a lower and an upper limit, both testing parameters must be determined.

If the testing parameter is greater than or equal to a constant Q (quality index) no penalty is applied to the contract sum in respect of the characteristic concerned.
If the criterion is not met, a penalty is imposed; its magnitude is dependent on the level of the result of the testing parameter; Q is a constant whose magnitude is dependent on the size of the random sample and on the uncertainty percentage which is still just considered permissible, as well as on the probability of approval (or probability of no penalty) applicable for example to a batch of products which is still just acceptable (in practice often 95%).

The Q values will be calculated with the aid of the following simple formulae determined by Stange [2]:

$$Q = \frac{|\xi_\delta|}{1 + \varepsilon} - \frac{\varepsilon}{|\xi_\delta|}, \text{ with } \varepsilon = \frac{|\xi_{1-\alpha}|}{\sqrt{2n}},$$

in which ξ_δ = ξ-value applicable to the maximum permissible error percentage (δ) or the maximum permissible percentage of 'bad' material.
$\xi_{1-\alpha}$ = ξ value applicable to the probability of approval or probability of no penalty for a batch with the maximum permissible percentage of 'bad' material; α is the threshold of unreliability; in practice a value of 0.05 is generally used in which case the probability of approval is 95% (n is the size of the random sample).
The percentages associated with the ξ values will be found in tabular form in standard books on statistics (see [3], page 43).
If ξ is negative, the table shows the fraction of observations smaller than R and if ξ is positive, the fraction of observations greater than R.
The formula for the testing parameters coincides well with the formula by which the percentage of observations in a normally distributed population which is smaller or larger than a given R value, is determined, i.e.

$$\frac{R - \mu}{\sigma} = \xi$$

The difference now is that the mean μ and standard deviation σ of the population are replaced by the mean $\bar{x}$ and standard deviation s of the random sample.
If a number n of cylindrical random samples are taken non-selectively from the same asphalt pavement, we shall naturally find, for the mean $\bar{x}$ and standard deviation s of e.g. the asphaltic bitumen content, values which not only differ among themselves but also depart in a more or less random manner from the 'true' mean bitumen content μ and the 'true' standard deviation σ.
As we have already seen, account must be taken of the influence of these random

deviations on the criteria concerned; this is done by using Stange's formulae referred to above for determination of the Q values.
However, these criteria can in theory only be applied if the random samples are taken from populations with normal or practically-normal distribution.
A population with a distribution which departs significantly from that of a normally distributed population while still having the same mean (μ) and standard deviation (σ) will have an uncertainty percentage which differs from the latter distribution. This may be a drawback for application of the criteria referred to above since the criteria concerned may become 'blurred' – with inevitable consequences for quality control. Research carried out by the American industry has, however, shown that there are seldom wide differences in uncertainty percentages between these distributions.
Therefore in the overwhelming majority of cases the error made in applying these criteria to populations which deviate in varying degrees from the normal distribution will not be important [4]. Figure 2 shows as examples four histograms for different *characteristics of layers of works chosen at random. It is apparent that the deviations* from the normal distribution remain within reasonable limits.

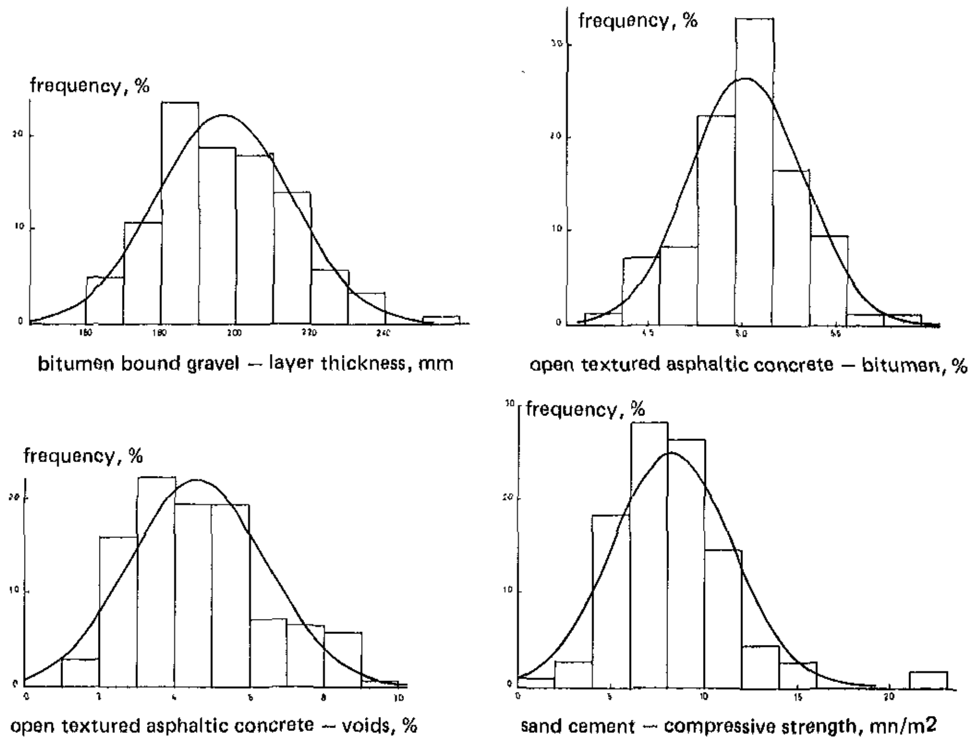

Figure 2. Histograms for certain characteristics of pavement layers chosen at random.

By using the criteria defined above we have obtained a test procedure in which 'batches' (in this case pavement layers) which have identical uncertainty percentages but differ in terms of mean and standard deviation, have the same probability of approval (or of avoiding penalty imposition) (Fig. 3); this is very important in the road building sector where the asphalt mix production, among other factors, may differ widely from one contractor to another.

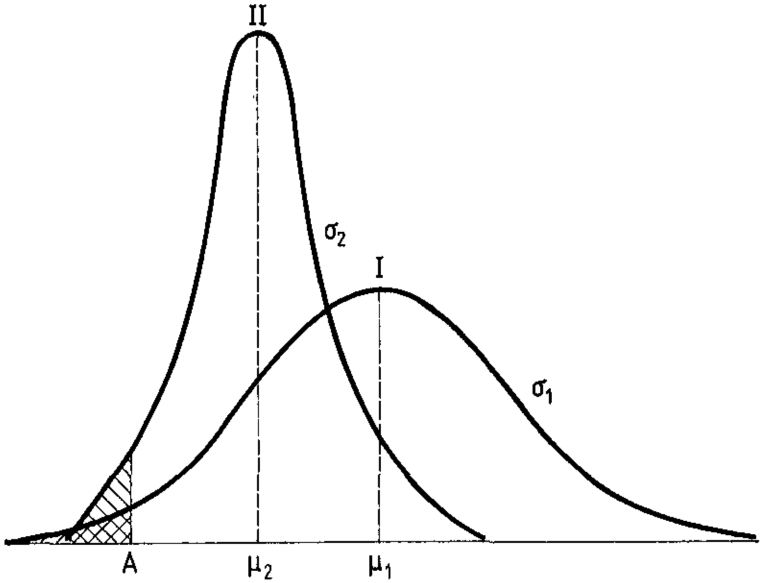

Figure 3. Two production processes which differ in terms of mean values and standard deviations but give the same percentage defectives.

Size of random sample

The size of the random samples chosen for statistical quality control of the various characteristics of asphalt pavements and sand cement roadbases is shown in Table 4 with reference to pavement area in m^2.

To ensure reasonably good differentiation between 'good' and 'bad' works, the random sample size per lot is set at 40 samples to determine the quality of the various characteristics of asphalt pavements and sand cement roadbases. Partly because determination of the asphaltic bitumen content of the different types of asphaltic concrete is extremely labour-intensive, and therefore expensive, a random sample size of 20 samples per lot is used for quality control of the asphaltic bitumen content; this number is necessary to obtain sufficient information on the quality of the completed work.

Table 4 Size of random samples for statistical quality control

characteristic	size of random sample with ref. to pavement area in m²		
	20,000–200,000 m²	200,000–400,000 m²	> 400,000 m²
asphaltic bitumen content of bitumen bound gravel, open-textured asphaltic concrete and dense asphaltic concrete	20	40	60
percentage voids of bitumen-bound gravel, open-textured asphaltic concrete and dense asphaltic concrete	40	80	120
degree of compaction of sand asphalt	40	80	120
compressive strength of sand cement	40	80	120
thickness of the different layers	40	80	120

Road works with surface areas between 200,000 and 400,000 m² are divided into two, and works with a surface area greater than 400,000 m² into three sections (lots) of identical size. All the lots are the subject of individual quality control. To determine asphaltic bitumen content, 20 samples are used for each lot while 40 samples per lot are used to determine the quality of the other characteristics shown in Table 4.

For projects with a surface area of less than 20,000 m², statistical quality control using this system is considered too expensive. The obvious solution here is to assess the quality of such works by investigating individual samples taken for example for each 1000 m²; conclusions can then be drawn from the results of these tests of individual samples.

In the case of small works, however, a better alternative will normally be to ensure frequent and thorough control by the public authority of the performance of work by the contractor.

The new system always uses individual samples (taken non-selectively), i.e. each sample consists of a cylinder with a diameter of 0.10 m taken from the pavement perpendicular to the road surface; in the previous system on the other hand the samples consisted of two cylinders with a diameter of 0.10 m obtained by drilling from the pavement on transverse lines at intervals of 1.0 m, perpendicular to the road surface. In the new system of acceptance control it is desirable to use single samples, since the sampling method might otherwise introduce inaccuracies into the random sample sizes [5].

New criteria for quality control of asphalt pavements and sand cement roadbases

In defining the criteria for quality control of asphalt pavements and sand cement roadbases use was made, for the various characteristics, of the corresponding mean of the random sample averages ($\bar{\bar{x}}$) and mean standard deviation ($\bar{s}$).
These values have been calculated with reference to the quality control results for several hundred projects in the years 1968-1978 (see Table 2, columns 3 and 4).
In these calculations, the values for $\bar{\bar{x}}$ and $\bar{s}$ only required modification in one single instance on the basis of practical data and information on desired and feasible quality levels. It was assumed that the requirements for the level (μ) and extent of dispersion (σ) (i.e. mean values calculated from the results not only of good but also of moderate and even poor works) must still be just acceptable to the directorate and considered reasonably feasible by the conctractor ('standard job'). This seems a better assumption than to take values for μ and σ calculated solely from the results of moderate and good works.
In determining the criteria on the basis explained above, the Q values were always defined in such a way that where the mean level and standard deviation for the various characteristics just met the required standards, there would be a 95% probability of no penalty in respect of each characteristic. This principle can be summarized as follows: in the newly developed system of acceptance control, the level and dispersion values used are taken from data for a large number of projects with the criteria fixed in such a way that when these values are respected the likelihood of a penalty is slight.

Composition and density of asphalt mixes and strength of sand cement

Columns 3 to 9 of Table 2 show the following data for the different characteristics:
- the values determined in the manner outlined above for the mean (μ) and standard deviation (σ) used – with reference to the results achieved in practice or the values prescribed for μ – as the basis for defining the criteria;
- the penalty or quality limits (R) now fixed on the basis of practical results and statistical considerations and representing, by definition, the boundary between 'good' or 'satisfactory' and 'bad' or 'not satisfactory' work;
- the ξ values calculated as $\frac{R - \mu}{\sigma}$ and the maximum permissible error percentages (δ) determined therefrom with the aid of tables, using the method of calculation described above (see [3], page 43);
- the subsequently determined Q value which shows that with the given random sample size (40 or 20) there is a 95% acceptance probability for a characteristic with the maximum permissible error percentage;
- the definitive Q value determined by rounding-off or fixed for reasons of uniformity.

The following explanation is appropriate here.
The Q value determined by rounding-off or fixed for reasons of uniformity functions in the general criterion as follows:

No penalty where

$$\frac{|R_{\text{min or max}} - \bar{x}|}{s} \geqq Q.$$

Penalty imposed where

$$\frac{|R_{\text{min or max}} - \bar{x}|}{s} < Q$$

In the latter instance the excess percentage associated with the calculated value for the penalty figure

$$\frac{|R_{\text{min or max}} - \bar{x}|}{s}$$

will be found in the table for the normal distribution [3]. For quality control of the asphaltic bitumen content of the different types of asphaltic concrete, a double criterion will be applied in the following manner:
No penalty where:

$$\frac{|R_{\text{min}} - \bar{x}|}{s} \geqq Q \quad \text{and} \quad \frac{R_{\text{max}} - \bar{x}}{s} \geqq Q$$

A penalty is on the other hand imposed where:

$$\frac{|R_{\text{min}} - \bar{x}|}{s} < Q \quad \text{and/or} \quad \frac{R_{\text{max}} - \bar{x}}{s} < Q$$

If in both cases the value of the test parameter is lower than Q, the corresponding excess percentages B_1 and B_2 must be added together.

Layer thickness

As regards the layer thickness, the penalty system is only applied for three separate thicknesses, i.e.
– the overall thickness of the different layers of asphaltic concrete, i.e. of the bitumen-

bound gravel, open-textured asphaltic concrete and frequently also dense asphaltic concrete which are of primary importance to the entire dimensional characteristics of the pavement;
– the thickness of the roadbase where it does not consist of bitumen-bound gravel (i.e. sand cement or sand asphalt) which plays a separate role in the dimensions of the pavement;
– the thickness of the temporary or permanent surface layer of the pavement with the emphasis placed on durability of this layer from the angle of direct mechanical stress and maximum particle size.

Apart from the pavement surface layer, the total thickness of the various layers of asphaltic concrete is also determined; it is therefore naturally possible to compensate at any time on the site a shortfall caused for one reason or another in the thickness of a particular layer, by applying additional thickness to one or more of the upper layers.
Statistical interpretation is effected wherever possible, i.e. in so far as the number of samples taken from the pavement or the results obtained by local measurement allow this. In cases (e.g. service roads, local roads, cycle tracks and access roads) where the number of cylindrical samples to be taken from the pavement is insufficient for statistical evaluation, the present system of penalties with testing of individual samples, is maintained in respect of layer thickness (see Table 1). The present system of penalties can also be applied to reconstruction projects where the nature of the work is such (wide variations in layer thickness) that statistical interpretation would be unrealistic.
To ensure convenient and uniform test criteria for the overall layer thickness of asphaltic concrete structures, the thickness of sand asphalt or sand cement roadbases and the thickness of temporary or permanent surface layers, the same criterion is applied in all possible cases (see Table 2) ($Q = 1.40$).

To ensure that, with the set Q value of 1.40, works with correct layer thickness (μ) (i.e. the layer thickness for which the contractor is to be paid), and a correct standard deviation σ, are 95% sure to avoid penalty with a random sample size of 40 core samples, it is possible, using the known basic data, to calculate the penalty limit R_{min} in the various cases from the following formulae (R_{min} being a completely arbitrary number):

$$1{,}40 = \frac{|\xi_\delta|}{1 + \varepsilon} - \frac{\varepsilon}{\xi_\delta} \quad \text{and} \quad \varepsilon = \frac{1{,}85}{\sqrt{80}} \quad \text{or:}$$

$\xi_\delta = 1{,}78$ (permissible uncertainty percentage: 3.8%)

$$\frac{R_{min} - \mu}{\sigma} = -\xi_\delta \quad \text{or} \quad R_{min} = \mu - 1{,}78\,\sigma$$

Table 2 shows penalty limits (R_{min}) calculated with the aid of these formulae for the overall thickness of the various layers of asphaltic concrete and for the thickness of the separate layers of dense asphaltic concrete, open-textured asphaltic concrete, sand asphalt and sand cement.

Atypical values

In incidental cases a value may occur (in a random sample of 20 or 40 results) which does not really seem to belong to the population. These values may for example fall outside the '3σ limit'. Such results which are attributable to a variety of causes and are not always due to careless work by the contractor, have a relatively large influence on the standard deviation (s) and thus also on the value of the test parameter

$$\frac{|R_{min} - \bar{x}|}{s} \text{ or } \frac{R_{max} - \bar{x}}{s}.$$

To give the contractor the benefit of the doubt, the following rule is applied:
If one (and not more than one) of the 20 measured results for the asphaltic bitumen content of the various types of asphaltic concrete or one of the 40 results for the voids content of the various types of asphaltic concrete, the degree of compaction of the sand asphalt or the compressive strength of the sand cement, fall outside the limit values shown below, this result will be treated as a value which does not belong to the population (and is therefore atypical: see Table 5).

Table 5 Limit values for atypical results

characteristic	limit value for atypical results
asphaltic bitumen content of bitumen-bound gravel, open-textured asphaltic concrete and dense asphaltic concrete	$V_b \pm 1.0\%$*
Voids of:	
bitumen-bound gravel	12%
open-textured asphaltic concrete	11%
dense asphaltic concrete	10%
degree of compaction of sand asphalt	92%
compressive strength of sand cement	1.0 MN/m^2

Application of the '3σ rule' leads, with some rounding-off, to these limit values; because of the relatively wide dispersion, the '3σ rule' is not applicable to the compressive strength of sand cement; an arbitrary limit value of 1.0 MN/m^2 is therefore taken, below which the test results can scarcely be considered reliable.

* V_b = prescribed value

Allowance for atypical values in the case of layer thickness control appears less appropriate for a variety of reasons, especially as the accuracy of a value which appears to be seriously inadequate can normally be verified by direct and objective means.

Test characteristics

Figures 4 to 6 inclusive show the operating characteristic curves in respect of the criteria defined above for assessing the bitumen content and voids percentage of the various kinds of asphaltic concrete, the degree of compaction of sand asphalt and the compressive strength of sand cement.
Details of the method for calculating these characteristic curves will be found in handbooks. The test characteristic shown in Figure 5 is also applicable to the acceptance criteria for total thickness of the different layers of asphaltic concrete, and for the layer thicknesses of dense and open-textured asphaltic concrete, sand asphalt and sand cement. These curves show the relationship between the probability of approval (P_g) or likelihood of no penalty, and the percentage of 'bad' material or the error percentage attibutable to a layer of asphaltic concrete, sand asphalt or sand cement. As regards the characteristic curve for the bitumen content of various types of asphaltic concrete (see Figure 4) it should be noted that precise determination is not possi-

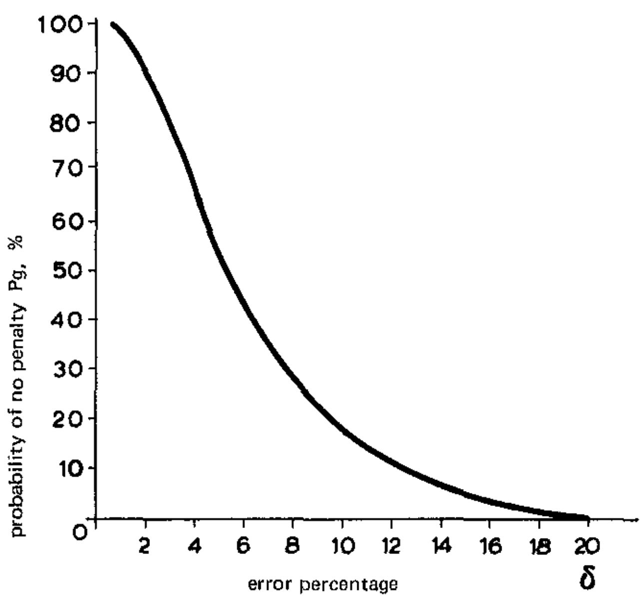

Figure 4. Characteristic curve for determination of asphaltic bitumen content of asphaltic concrete.

ble, partly because of the fact that a dual criterion is applied for the determination of the asphaltic bitumen content. In particular for relatively low error percentages, between 0.5 and 1.5%, the probability of no penalty being imposed will be somewhat lower than the characteristic curve suggests.

The different figures clearly show that penalties will only be incurred in exceptional cases by good work while poor work will almost always be penalized. For example, if the error percentage in respect of the voids content of a layer of bitumen-bound gravel is 1%, the likelihood of no penalty being imposed is 99.9%; if on the other hand the error percentage is 8% (for example for work with a mean value of 6.7% and a standard deviation of 2.0%), the likelihood of penalties being justifiably imposed is very high, i.e. in the order of 80% (see Figure 5).

As we have already seen, a penalty may exceptionally be imposed on the contract sum where the work is in fact good; this is known as the producer's risk. On the other hand there is also a consumer's risk: it may happen, although again this is exceptional, that no penalty is imposed on work which is definitely bad. For example, if the error percentage in respect of the voids content of a layer of bitumen bound gravel is 10% (e.g. for work with a mean value of 6.9% and a standard deviation of 2.0%), the likelihood of no penalty is 9%; in other words 1 out of 11 such works will escape any penalty. It should of course be noted that in the statistical method of quality control of a road project, individual characteristics are not assessed separately; on the contrary a number (generally about 10) of characteristic parameters (bitumen content and density of asphalt, strength of sand cement, layer thicknesses of 3 or 4 different pavement layers) are determined.

This in turn means that where a penalty is wrongly imposed on good work or alternatively no penalty incurred by bad work, the error will only apply to one part of the work and never to the work as a whole so that the consequences which may arise in exceptional cases affect only that particular part of the work and never the project in its entirety.

In determining the various criteria for the purpose of acceptance control, the analysis is in all cases based on values for the level and extent of dispersion which the contractor can reasonably be expected to meet.

It is open to question whether, in the event of reduction of the minimum standards placed on the different characteristics, the work may still just be acceptable. However, a reduction of these minimum requirements necessarily entails a higher error percentage which increases the consumer's risk; in that event the likelihood of poor work escaping any penalty is higher. From the standpoint of good quality control, a reduction of the set minimum standards must therefore be treated as impermissible. Calculations have also been made to determine the approximate percentage probability of relatively low or high excess percentages in respect of certain mean values (μ) and standard deviations (σ) for the various characteristics.

For details of the calculation of these probabilities, reference should be made to standard statistical handbooks. The results of the calculations show that if, in excep-

tional cases, a penalty is charged on good work that penalty will be relatively low, i.e. less than 2 to $2\frac{1}{2}\%$ of the total cost of the layer, since the excess percentage will practically never be higher than the 10% allowed for the asphaltic bitumen and voids content for the various kinds of asphaltic concrete and the 15% for the degree of compaction of sand asphalt and compressive strength of sand cement. Again in the case of works which are slightly below the minimum requirements for level and dispersion, the likelihood of an excess percentage (depending on the characteristic) of more than 10% or 15% is extremely low.
A high probability of an excess percentage of more than 25% in fact only occurs – and then quite rightly – in the case of bad work with error percentages of 20% or more.
Considerable likelihood of excess percentages between 15 and 25% for asphaltic bitumen content and voids content occurs in the case of works which fall distinctly short of the set minimum standards relating to mean and standard deviation. In general these are works with error percentages between 12 and 20%.

Penalty criteria

When these criteria are applied it is reasonable to assume that this statistical system is less stringent than the system applied hitherto, especially for percentages which are only marginally in excess of the penalty limits. It has been found in practice that penalties on many works fall precisely within this area.
Critical consideration of the relationship between shortfall on quality and the resulting, desirable compensation charges or penalties, leads us to conclude that penalties under the present system are in fact relatively low.

Penalties

In the light primarily of experience of the non-statistical quality control system used during the past ten years, the relationship between the scale of the penalties K and the percentage of 'unsatisfactory' or 'poor' test results B in the statistical system has been determined as follows:

$K = 0.3\ B - 1.0$ for bitumen content and voids content of asphaltic concrete, and
$K = 0.3\ B - 2.0$ for layer thicknesses, degree of compaction of sand asphalt and compressive strength of sand cement.

K is expressed here as a percentage of the true value of the layer concerned, while B is the percentage of the overall work which, on the basis of the calculation (from the mean value $\bar{x}$ and standard deviation s), can be considered to fall below the prescribed quality (penalty) limit R in respect of a particular characteristic (B = '% defectives').

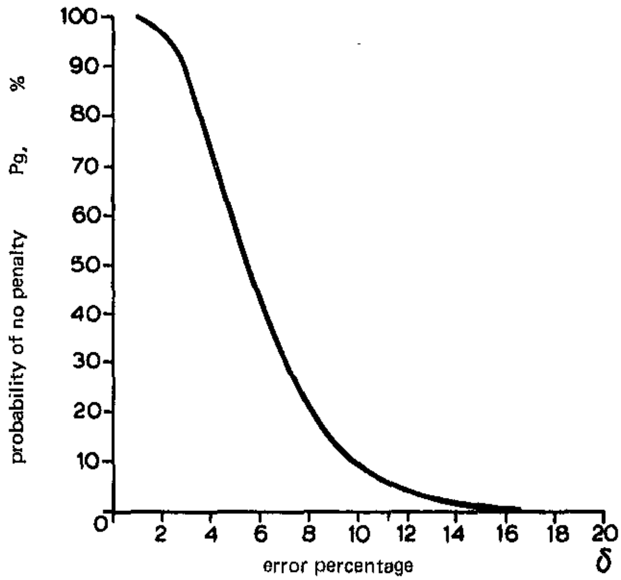

Figure 5. Characteristic curve for determination of voids content of asphaltic concrete.

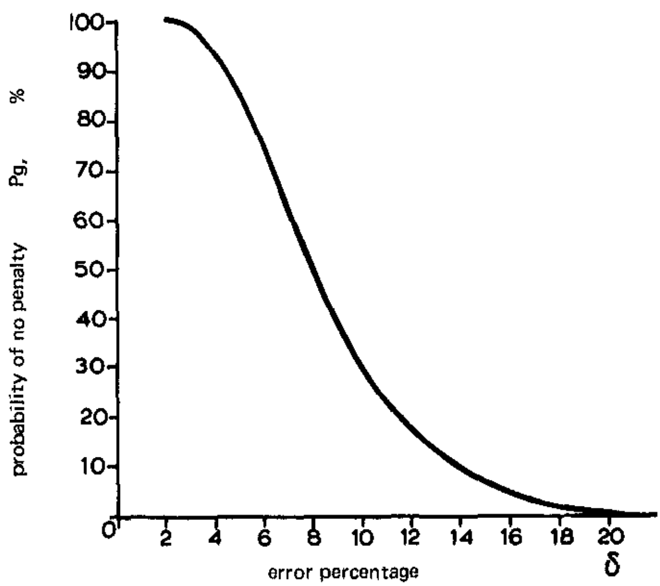

Figure 6. Characteristic curve for determination of degree of compaction of sand asphalt and compressive strength of sand cement.

Introduction of the new statistical system

In discussions with the committee of experts of the road building contractors organizations, a wish was expressed for more information on the consequences of the new system before it was actually introduced.
It was agreed that as an initial step a number of projects (10 to 20) would be assessed by both systems, i.e. the traditional non-statistical and the new statistical system.
The results would then be compared and discussed with a view to possible adjustment of the statistical system.

Costs and benefits

It must be remembered that neither quality control during implementation nor acceptance control can in themselves give complete certainty as to the overall average quality of a particular project or prevent instances of poor quality. Investigations are and remain based on arbitrary, random sample checks. It also practically impossible to obtain optimum quality at the lowest possible cost to the authority which has commissioned the works.
The aim of the penalty provisions cannot therefore be primarily to provide equitable compensation for lower quality. All rules laid down on a largely theoretical basis with that aim in mind would be influenced by so many factors (subsoil, traffic load, maintenance methods, wheather conditions etc.) that a precise approximation is impossible. The introduction of an apparently watertight system based on cost-benefit analysis, would therefore be inequitable to the contractors who perform the work.
It is, however, particularly important to have some knowledge of the costs and benefits arising in this context. We refer to the costs resulting from a shorter useful life due to poor quality, and to the benefits flowing from the proceeds of the penalties imposed because of quality shortcomings.

A special study of this problem was made by Brouwers in 1974; it was hitherto only available in Dutch [10], but now appears as chapter 4 of this publication.

Control of surface characteristics

An important area of acceptance control relates to the surface characteristics of evenness and skid-resistance. Measurements of evenness and skid resistance are made by methods which are not entirely statistical. The fact that it is not easy to express traffic safety in statistical figures plays a part here. In the first instance only 30% of the road length is checked for evenness and skid-resistance in order to limit the number of measurements. The location of the measuring points is fixed at random. However, if the results of this first series of measurements, and the appearance of the surface, give reason to suppose that there are large-scale shortcomings on quality, especially as regards skid-resistance, further measurements are carried out; should this be necessary, the entire surface of the completed work is controlled.

Evenness

Untill 1975 the evenness was always checked by using a normal or rolling straightedge with a length of 3 m. When deviations from the even profile of more than 3 mm were found in a measured section with a length of 100 m, penalties were imposed. If deviations of more than 5 mm were found more than once, the evenness had to be corrected by shaving – naturally at the contractor's cost. Since 1976 the viagraph has been used instead of the rolling straightedge to check the evenness of the carriageway surface. A penalty is imposed if the deviation percentage C_5 is greater than 2. Deviations of more than 5 mm, measured with the viagraph, necessitate correction by shaving at the responsibility of the contractor concerned. In special cases where the irregularity is such that correction by shaving will not result in a completely even surface, the contractor is required by special contract provisions to adjust the surface by laying an extra surface course of 40 mm on the completed road surface.

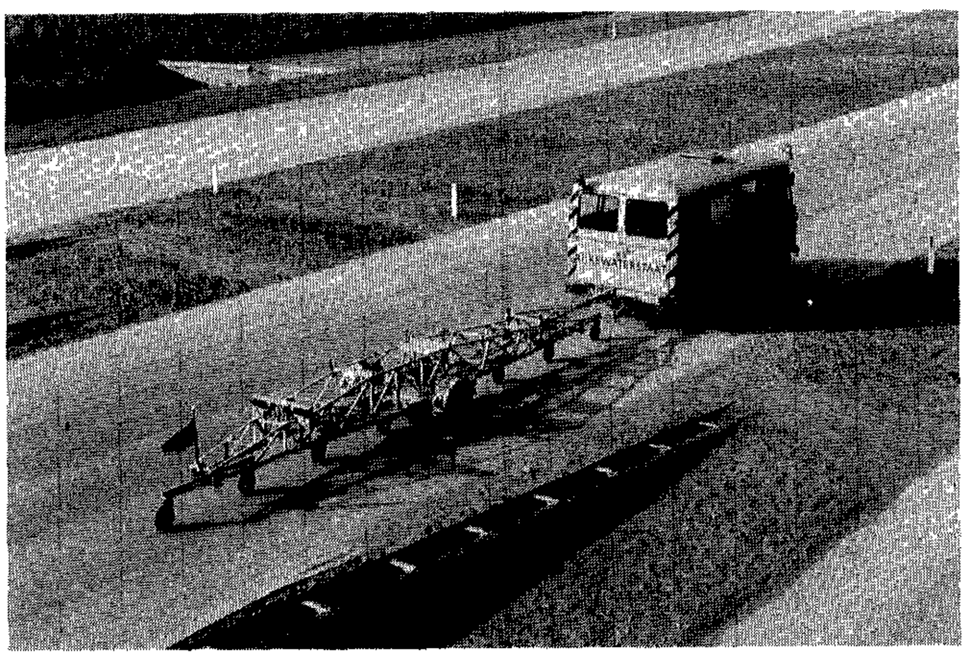

Figure 7. Viagraph for measuring evenness of road surfaces.

Skid-resistance

Since as long ago as 1967 it has been stipulated in the Netherlands that before trunk roads are opened to traffic, new pavements must be checked for skid-resistance. This is done with a standard measuring vehicle with a retarded wheel (86% slip), a wet surface and a measuring speed of 50 km/h. Penalties are imposed if the measured coefficient of friction is lower than 0.52.* However, if the value is less than 0.45* the contractor must also correct the surface of the asphaltic concrete by treatment with white spirit and crushing sand. This treatment adequately removes surplus bitumen from the surface until a coefficient of friction of at least 0.52 is obtained. However, it is very expensive and the provision in the contract thus has a good deterrent effect. New roads with smooth surfaces are thus rarely encountered today and the completed surface normally easily meets the requirement regarding skid-resistance. Other methods of improvement have also been applied recently, i.e. sand-blasting or milling.

Figure 8. Standard vehicle for measuring skid resistance of road surfaces.

* These limit values are specified since January 1978. With respect to international standardization of skid-resistance test tyres as from 1978 the measuring vehicles of the SRL have been provided with standard test tyres of a type different from the one used since 1958.
Starting from the original criteria the numerical values for the specifications on skid-resistance of road surfaces have been changed on the basis of comparative measurements with both types of tyres. Before 1978 the specified limits were 0,56 and 0,51 instead of 0,52 and 0,45 respectively.

One stipulation is particularly important in this connection. For more than ten years Dutch road contract specifications have stated that warm asphaltic concrete surface layers must be spread during rolling with approx. 2 kg fine stone chippings (2-5 mm) per m^2. This treatment effectively prevents initial smoothness of new asphaltic concrete surfaces due to excess bitumen content in the surface.

4 Relationship between the penalty system and the necessary compensation costs

In this final chapter calculations are given for the layer thickness, voids and asphaltic bitumen content of an asphalt pavement; these calculations show that the penalties are substantially lower than the compensation costs. In the case of the layer thickness reference is made to a pavement structure which is frequently used in practice and the consequences of divergent layer thickness distributions (both too low average thickness and excessive standard deviations) on the service life of the pavement are calculated. Comparisons are drawn with the average service life of the standard pavement structure determined by a design method.
Similar calculations are effected for the voids and bitumen content; the reduction in service life can again be calculated with reference to an asphalt structure with voids equivalent to the mean voids content of all works to which penalties have been applied, or with a bitumen content equivalent to the desired bitumen content.

The following formulae are used for calculation of the penalties:

$K = 0.2\ B$ for the layer thickness;
$K = 0.3\ B$ for asphaltic bitumen content and voids.

When this study was carried out, about one year before development of the statistical system described above was fully completed, it was not yet known which specific formulae would be proposed in the final version. Although the latter do in fact differ somewhat from the formulae reported earlier (i.e. $K = 0.3B - 2.0$ and $K = 0.3B - 1.0$) the original formulae have been maintained in this chapter, as Tables 3 to 7 would otherwise have had to be revised. The differences concerned are only slight so that this has no great influence on the conclusions drawn.

Layer thickness

The damage resulting from insufficient layer thickness can be quantified on the basis of design data establishing a relationship between the layer thickness and traffic parameters. Use is made below of the 'design formula of the AASHO test' which establishes the relationship between the number of load repetitions n up to the attainment of a given serviceability index p, and the thickness, expressed as the thickness index D for a given wheel load P (tons). This formula, valid for $p = 1.5$ – a low level of, in particular, longitudinal evenness of the carriageway at which reconstruction becomes necessary –, is as follows:

$$n = \frac{10^{5.93}(D + 1)^{9.36}}{(4.41\,P + 1)^{4.79}}$$

In this formula $D = \Sigma c_i h_i = c_1 h_1 + c_2 h_2 + \dots$, an equivalent pavement thickness formed by summation of the layer thicknesses of the different pavement layers (h in cm), each multiplied by the value coefficient c.
For the reference wheel load $P = 5$ tons, this formula becomes

$$n = 0.2528(D + 1)^{9.36}$$

or $D = 1.1583n^{0.107} - 1$, where n is the equivalent number of load repetitions of the 10 ton axle load (or equivalent 5 ton wheel load). This design formula naturally only applies under conditions similar to those for the AASHO test as regards the underlying subsoil and the environment.

For a more detailed description of the AASHO test reference should be made to specialized publications, e.g. the paper by Van de Fliert and Brouwers [9].
The example chosen is the standard structure of a primary road (non-motorway) based on:

4 cm coarse dense asphaltic concrete	with $c = 0.173$
4 cm open-textured asphaltic concrete	
18 cm (3 × 6 cm) bitumen-bound gravel	with $c = 0.12$

There are thus 26 cm asphaltic concrete in all (nominal layer thickness) applied to a compacted subgrade with a thickness of 50 cm and $c = 0.02$.
The thickness index of this structure is
$D = 8 \times 0.173 + 18 \times 0.12 + 50 \times 0.02 = 4.55$
On the basis of the invoicing clauses in contract specifications with penalty provisions, the effective mean layer thickness of the bituminous structure is as follows with a mean unit density (normal value) of 2.33:

$\frac{2.50}{2.33} \cdot 4 = 4.3$ cm dense asphaltic concrete

$\frac{2.50}{2.33} \cdot 4 = 4.3$ cm open-textured asphaltic concrete

$\frac{2.50}{2.33} \cdot 18 = 19.3$ cm bitumen-bound gravel

Total = 28 cm asphaltic concrete.

The thickness index of the average structure is therefore:
$D = 8.6 \cdot 0.173 + 19.3 \cdot 0.12 + 50 \cdot 0.02 = 4.80$
On this pavement with $D = 4.80$, the dimension formula shows that $n = 3.536 \cdot 10^6$ (equivalent 10 ton axle) load repetitions are permissible before the serviceability index $p = 1.5$ is reached.
The service life of this pavement in years can be calculated on the basis of two further factors: the number of equivalent 10 ton load repetitions in the year of construction and the annual percentage growth in this number. We have taken for these factors $n_o = 84{,}000$ and $\alpha = 7\%$ respectively, i.e. an equivalent annual total and a mean growth percentage which have recently been used as the characteristic figures for primary roads.
The service life in years can now be claculated (for example) graphically with the aid of a graph showing the cumulative total of axle loads as a function of the service life in years for the applicable (continuous) growth percentage. It is thus calculated that the structure under consideration with $\Sigma n_{eq5t} = 3.536 \cdot 10^6$ has a service life T of 20.3 years. On the basis of the relationship $D = \Sigma c_i h_i = f(n)_{p,P}$ and $T = f(n)_{n_o,\alpha}$ i.e. $D = f(T)$, the service life in years can now be calculated for every pavement thickness. The reduction in layer thickness proportional to the design thicknesses is distributed over the asphalt layers so that with the asphalt thickness H in cm:

$$D = (4.80 - 1.00)\frac{H}{28} + 1.00.$$

On the basis of a normal population distribution, the layer thickness of the pavement is entirely determined by the mean layer thickness μ and standard deviation σ. 28 cm is taken as the feasible average for this 'normal' structure; this is the layer thickness which, in the closest possible approximation to the calculated daily quantity of 25 kg/m^2 per cm layer thickness, can be expected for asphalt with a relatively high unit density. 2.0 cm is taken as the feasible standard deviation for the total layer thickness. Both assumptions can be considered realistic on the basis of the results obtained. It is now also easy to show, on the basis of the normal distribution, the dispersion in layer thickness over this total 'standard' pavement and hence to express the dispersion in terms of service life.

The calculations have been compiled in Table 6.

The significance of the 'nominal layer thickness' is now clearly apparent: 84.1% of the pavement surface has a thickness equal to or greater than the nominal thickness of 26 cm, and therefore a service life equal to or greater than 16 years.
It is also more in accordance with road-building practice to take as the effective service life of the pavement the life at the nominal layer thickness rather than at the mean layer thickness. Premature maintenance of the pavement will certainly be necessary

Table 6 Dispersion in layer thickness of the 'standard' pavement and service life of parts of the pavement

asphalt pavement layer thickness H, cm	pavement surface, % of total (Q)		service life of pavement: asphalt layer thickness, cm	D	equivalent no. 10 t axles	years, T
> 28	50		> 28	> 4.80	$> 3{,}536 \cdot 10^6$	> 20.3
27 – 28	19.1	84.1	28	4.80	$3{,}536 \cdot 10^6$	20.3 (20)
26 – 27	15.0		27	4.664	$2{,}833 \cdot 10^6$	18
25 – 26	9.2		26	4.529	$2{,}258 \cdot 10^6$	15.7 (16)
24 – 25	4.4		25	4.393	$1{,}789 \cdot 10^6$	13.5
23 – 24	1.7		24	4.257	$1{,}409 \cdot 10^6$	11.5
22 – 23	0.5		23	4.121	$1{,}103 \cdot 10^6$	9.6 (9.5)
< 22	0.1		22	3.986	$8{,}580 \cdot 10^5$	8
			21	3.850	$6{,}627 \cdot 10^5$	6.5
			20	3.714	$5{,}081 \cdot 10^5$	5.2 (5)
			19			4

Table 7 Layer thickness dispersion of 5 pavements with excessively low mean thickness or high standard deviation and effective life of these pavements

variant	I		II		III		IV		V	
mean layer thickness μ, cm	27		26		24		28		28	
standard deviation σ, cm	2		2		2		3		4	
% of pavement surface with layer thickness:										
> 28 cm	30.9		15.9				50.0		50.0	
27 – 28 cm (= 28)	19.1	84.1	15.0				12.9	84.1	9.9	
26 – 27 cm (= 27)	19.1		19.1	84.1	15.9		11.9		9.2	84.1
25 – 26 cm (= 26)	15.0		19.1		15.0		9.3		8.2	
24 – 25 cm (= 25)	9.2		15.0		19.1	84.1	6.7		6.8	
23 – 24 cm (= 24)	4.4		9.2		19.1		4.4		5.3	
22 – 23 cm (= 23)	1.7		4.4		15.0		2.5		3.9	
21 – 22 cm (= 22)	0.5		1.7		9.2		1.3		2.6	
20 – 21 cm (= 21)	0.1		0.5		4.4		0.6		1.8	
19 – 20 cm (= 20)			0.1		1.7				1.1	
< 19 cm (= 19)					0.6				0	
effective layer thickness, cm	25		24		22		25		24	
effective life T, years	13.5		11.5		8		13.5		11.5	

for that part of the pavement which is thinner than 25 cm, i.e. 6.7% of the total pavement area.

In the following analysis, it is always assumed that the greatest layer thickness of the thickness ranges shown in the table is the determining factor as far as service life is concerned. It is now possible to determine this life distribution for pavements with other layer thickness distributions, i.e. different mean layer thickness values μ and standard deviations σ, and, on the basis of a comparison with the standard structure, to determine the cost incurred for premature reconstruction and maintenance.

For this purpose the same distribution over the layer thickness groups shown in the table must be calculated, the service life of 8.41% of the pavement surface being taken as the effective service life.

The calculations (see table 7) were made for five variants with mean layer thickness deviation μ and standard deviation σ.

For the decisive part of the six construction variants taken from tables 6 and 7, Figure 9 shows the (standard) distributions of the pavement surface as a function of the layer

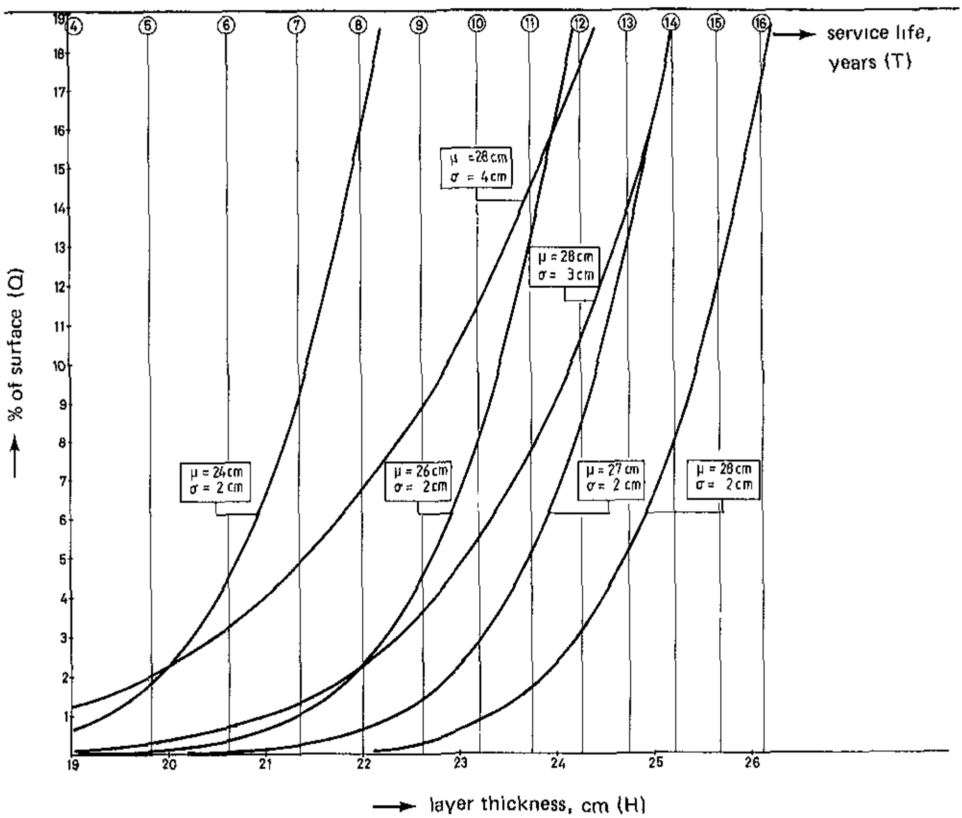

Figure 9. Distribution of road surface as a function of layer thickness and service life.

thickness; the service life calculated from the layer thickness is also shown.
The following assumptions are made for the purpose of calculating the cost of premature reconstruction and maintenance:
At the end of the effective life (T years), an asphalt layer must be laid down; the cost of this work is set at $f\,A$ per m^2 road surface.
In the normal structure, this asphalt layer will serve for an (extended) service life of 10 years (i.e. $16 + 10 = 26$ years after laying the pavement).

Since the effective life of the structure is only T' years instead of T years, this asphalt layer must be applied $(T - T')$ years sooner. This asphalt layer must have a greater thickness than under normal circumstances so that the service life is extended to $10 + (T - T')$ years.
At that point in time the original situation is restored. The updated cost per m^2 is therefore:

$$\frac{(1 + j)^{T-T'} - 1}{(1 + j)^{T}} \cdot A + \frac{B}{(1 + j)^{T'}}$$

j = discount rate equated with the interest rate on long-term public notes; a figure of 0.08 (8%) is used.
A = cost of a 4 cm asphalt layer; the total cost (inclusive of ancillary charges) is set at $f\,8/m^2$ road surface.
T = effective life of the extended structure, i.e. 16 years in this example.
The first term is the amount that must now be earmarked to apply the asphalt layer $(T - T')$ years sooner than normal.
B = cost of the extra thickness required for the asphalt layer per m^2 to ensure that it has a life of $(T - T')$ years longer than 10 years.
The cost of the extra layer thickness of the asphalt layer to be applied after T' years to ensure a service life of $10 + (T - T')$ years is calculated on the basis of a linear relationship between service life and layer thickness, so that:

$$B = \frac{16 - T'}{10} \cdot 4 \text{ cm}$$

(rounded off to whole cm).
Although this linearity – on the basis of design relationships (in effect reconstruction) – in fact results in excessive additional thickness, it must be remembered that the application of the asphalt layer also takes place by reason of factors which are not related to design and tend rather to be proportional to n (the 'improvement' is a hybrid of maintenance and actual reconstruction for design reasons).
The second term is thus the amount that must now be earmarked for the additional layer thickness to be applied after T' years. Finally the costs are expressed as a per-

centage of the total pavement costs set at f 20.–, so that the following improvement costs to be compensated are arrived at:

$$\left\{\frac{[(1.08)^{16-T'} - 1] \cdot 8}{1.08^{16}} + \frac{\dfrac{16 - T'}{10} \cdot 4}{1.08^{T'}}\right\}\frac{100}{20}\%$$

In addition to these costs necessitated by premature improvement, the cost of the associated increase in maintenance can also be assessed, again in comparison with a normal structure.

For this purpose, the difference in pavement area (as % of the total) must be calculated on the basis of the tables for the various thickness and service life groups. Additional costs will therefore be incurred for this purpose at the end of the effective life. Assuming that these costs are identical to the cost of normal reconstruction, i.e. f 8 per m^2 – which is plausible given that the non-selective distribution of the damage often entails the improvement of a substantially greater road surface area than is strictly necessary – the following percentage of the total pavement costs is arrived at:

$$\frac{8}{20}\sum\frac{Q - Q_0}{1.08^T}\ \text{percent}$$

Q is the percentage of the surface area in the layer thickness group for which maintenance is necessary prior to reconstruction. Q_0 is the percentage in the same layer thickness group for a normal structure.

The calculations for both amounts in respect of the five variants are shown below:

Structure I ($\mu = 27$ cm, $\sigma = 2$ cm)

$$\text{improvement: } \left\{\frac{(1.08^{2.5} - 1)8}{1.08^{16}} + \frac{1}{1.08^{13.5}}\right\}\frac{100}{20} = 4.1\%$$

$$\text{maintenance: } \frac{8}{20}\left\{\frac{4.4 - 1.7}{1.08^{11.5}} + \frac{1.7 - 0.5}{1.08^{9.5}} + \frac{0.6 - 0.1}{1.08^{8}}\right\} = 0.8\%$$

total 4.9% of pavement costs

Structure II ($\mu = 26$ cm, $\sigma = 2$ cm)

$$\text{improvement: } \left\{\frac{(1.08^{4.5} - 1)8}{1.08^{16}} + \frac{2}{1.08^{11.5}}\right\}\frac{100}{20} = 9.1\%$$

$$\text{maintenance: } \frac{8}{20}\left\{\frac{4.4-0.5}{1.08^{9.5}}+\frac{1.7-0.1}{1.08^{8}}+\frac{0.6}{1.08^{6.5}}\right\} = 1.2\%$$

total 10.3% of pavement costs

Structure III ($\mu = 24$ cm, $\sigma = 2$ cm)

$$\textit{improvement: } \left\{\frac{(1.08^{8}-1)8}{1.08^{16}}+\frac{3}{1.08^{8}}\right\}\frac{100}{20} = 18.1\%$$

$$\text{maintenance: } \frac{8}{20}\left\{\frac{4.4}{1.08^{6.5}}+\frac{1.7}{1.08^{5}}+\frac{0.6}{1.08^{4}}\right\} = 1.75\%$$

total 19.9% of pavement costs

N.B. As an alternative form of improvement, the application of a new 4 cm surface layer can be calculated; the cost is then:

$$\frac{100}{1.08^{8}} \cdot \frac{8}{20} = 24\%$$

It must be remembered that this is an additional operation which should not have been necessary and in the case of which allowance must also be made for compensation in respect of prejudice to the road user and road authority.

Structure IV ($\mu = 28$ cm, $\sigma = 3$ cm)
improvement: as structure I, 4.1%

$$\text{maintenance: } \frac{8}{20}\left\{\frac{4.4-1.7}{1.08^{11.5}}+\frac{2.5-0.5}{1.08^{9.5}}+\frac{1.3-0.1}{1.08^{8}}+\frac{0.6}{1.08^{6.5}}+\right.$$
$$\left.+\frac{0.4}{1.08^{5}}\right\} = 1.4\%$$

total 5.5% of pavement costs

Structure V ($\mu = 28$ cm, $\sigma = 4$ cm)
improvement: as structure II, 9.1%

$$\text{maintenance}\quad \frac{8}{20}\left\{\frac{3.9-0.5}{1.08^{9.5}}+\frac{2.6-0.1}{1.08^{8}}+\frac{1.8}{1.06^{6}}+\frac{1.1}{1.06^{5}}+\frac{0.6}{1.06^{4}}+\right.$$

$$\left.+\frac{0.5}{1.06^{3}}\right\}=2.3\%$$

total 11.4% of pavement costs

Finally a comparison can be drawn between the costs calculated in this way for compensation of deviant layer thicknesses (in terms of μ and σ) and the penalties determined on the basis of the penalty provisions (see table 8).
The penalty is then calculated on the basis of the mean layer thickness $\bar{x}$ and the standard deviation s of the random sample. It should be noted that in this way the excess percentage of the penalty limits is calculated using estimates, calculated on the basis of a random sample, of the mean standard deviation ($\bar{x}$ and s).
Although these parameters must approximate as closely as possible to the true mean μ and true standard deviation σ, there may in fact always be a disparity between them. The calculation is effected on the basis of the relationship:
Penalty (calculated as percentage of total pavement costs) = 0.2 × excess percentage in relation to penalty limit B, calculated from $\left|\frac{\bar{x}-R}{s}\right|$
the limit value R being fixed at 24.0 cm.

Table 8 Comparison between penalties and compensation costs for total pavement thickness

structure	$\frac{\mu - R}{\sigma}$	%defectives = B	penalty $0.2B$, % cost of pavement	calculated necessary compensation costs	cost/benefit ratio
normal	$\frac{28-24}{2}=2.0$	2.3	zero*	zero	–
I	$\frac{27-24}{2}=1.5$	6.7	1.34	4.9	3.7
II	$\frac{26-24}{2}=1.0$	15.9	3.18	10.3	3.2
III	$\frac{24-24}{2}=0$	50	10	19.9	2.0
IV	$\frac{28-24}{3}=1.33$	9.2	1.84	5.5	3.0
V	$\frac{28-24}{4}=1.0$	15.9	3.18	11.4	3.2

* zero = no penalty because excess percentage is greater than a set limit value [8]

For details of this factor 0.2 and the penalty limit referred to here for the total layer thickness, reference should be made to the relevant comments on page 32 of this publication and to publication [8].

With the exception of the extreme case of structure III, a fairly constant relationship is thus found between the costs and 'estimates', i.e. on average 1 : 3.
This means that the formula based on the deviation percentage for the total layer thickness is a good indication of the actual consequences of improvement of the pavement life.
In the relationship: penalty = factor × percent defectives, the factor must, however, be at least 0.6 to obtain adequate compensation.
The dimension formula used above is a special instance of a general dimension formula derived as follows from the AASHO results:

$$D = \Sigma\, c_i h_i = 0.004\, \Sigma\, E_i^{\frac{1}{3}} h_i = \left(\frac{250}{E_s}\right)^{0.4}\left[1.158\left\{\Sigma n_i \left(\frac{P_i}{5}\right)^{4.2}\right\}^{0.107} - 1\right]$$

D = necessary layer thickness expressed as thickness index D, up to $p = 1.5$
E_s = modulus of elasticity of subgrade
E_i = modulus of elasticity of pavement courses
h_i = layer thickness of pavement courses in cm
n_i = number of load repetitions of wheel load P_i during the service life (up to $p = 1.5$) of the road.

All in all, a safety coefficient of 0.11 $(D + l)$ is introduced on the basis of the 90% reliability limits of the AASHO relationship and a combination with the (dynamic) modulus of elasticity (= 100 × CBR) of the subgrade in the AASHO test (CBR = 2.5).
In the first approximation, we have taken a proportionality of the value coefficient of a pavement material (of thickness h cm) with the modulus of elasticity to the power of 1/3.

For the purpose of comparative calculations, this formula is further simplified [11] to

$$D = K(\mathrm{CBR})^{-0.4}(\Sigma n_i P_i^{4})^{\frac{1}{9}}$$

Although an infinite number of calculation examples can be worked out on the basis of this formula and on the basis of other design methods, a service life equation will always give broadly similar results, since these are based primarily on the low exponent of n while the subgrade characteristic which is introduced has a considerable influence on the absolute value of D but no influence at all on the relationship between the thickness indices for the same subgrade.

The parameter values chosen as an example are adapted to practical conditions and the use of a low determining CBR value (AASHO-CBR) in combination with reference to the topmost, wellcompacted layer of the subgrade, generally provides a sufficiently high safety coefficient to justify use of the original AASHO equation.

Finally it should be noted that the layer thickness variation of the total pavement has been treated in the foregoing as the determining factor for service life. Although variation both of the mean layer thickness and of the standard deviation has been taken into account, it seems reasonable in practice to take steps first of all to prevent excessive variation of the standard deviation. The provision that only the actually processed quantity can be invoiced – up to a maximum of 25 kg/cm per m^2 – is in itself a deterrent to excessively low average layer thicknesses.
Under normal circumstances processing of 'too little' asphalt will cut the contractor's profit.
It will also be expedient to lay down requirements for the thickness of the individual pavement layers, with reference to both the mean and standard deviation.
The greatest need for this applies to the topmost pavement layer and to the subbase, especially where this consists of a different material than the roadbase, e.g. sand cement.
For the top pavement layer a mean thickness of 4.3 cm has been calculated and a reasonable standard deviation is 0.5 cm.
With a mean layer thickness of 3.8 cm the total pavement thickness will be 0.5 cm lower, thus reducing the thickness index by 0.087 and leading to a reduction in survice life of about 1.5 years in comparison with a normal pavement.
Expressed as a percentage of the cost of the surface asphalt layer, compensation for premature improvement will therefore be:

$$\left\{\frac{(1.08^{1.5} - 1)8}{1.08^{16}} + \frac{1}{1.08^{14.5}}\right\}\frac{100}{4} = 15\%$$

or 3%, expressed as a percentage of the total pavement costs.

On the basis of the choice of $R = 3.3$ cm, $\dfrac{\mu - R}{\sigma} = 1.0$ so that the excess percentage in this case is 15.9% and the penalty with a factor of 0.2 is equal to only 3.2% of the value of the asphalt layer or 0.6% of the cost of the asphalt pavement.

However, the performance of these calculations is far less logical in this case since a shortfall on the thickness of an underlying layer can be compensated by additional thickness of another layer, while a penalty is already imposed for shortfalls (although much greater in extent) on the (total) layer thickness. The above considerations show that there can be no question of an accumulated penalty exceeding the damage actually caused.

The main purpose of penalties in respect of the layer thickness of the surface layer is in any case to avoid excessive standard deviations so that the design calculations lend themselves far less to use as a basis for comparison. The harmful consequences of local shortfalls on thickness of the surface layers are sufficiently well known from practical experience.
For comparison, it should be noted that the deviation percentage of 15.9% calculated in this case will also be reached with a standard deviation of 1.0 cm if the layer thickness (as is usual in practice) coincides with the mean thickness. This means that the surface layer with a mean thickness of 4.3 cm (nominal thickness 4 cm) has a thickness of less than 3.0 cm over 10% of the road surface and of less than 2.5 cm over 3.6% of the surface.

Voids

A reduction in service life is the quantifiable expression of damage caused by excessive voids content. The calculation can be based on fatigue tests which have been carried out on asphalt bars by a number of research workers. In 1960, Saal and Pell [12] already described fatigue tests (constant $\sigma = 3 MN/m^2$, 0°C, 50Hz) on sand asphalt bars (85% sand, 15% limestone filler, $9\frac{1}{2}$% (m/m) bitumen 40/50 M.E.). The voids content varied between 0 and 10% (V/V). They reported that the service life with 10% voids is only 1/16 of the life with 0% voids. The reduction is greater than would be expected from the lower quantity of asphalt in the cross-section so that the cause must lie in stress concentration.
Bazin and Saunier [13] also performed fatigue tests on a sand asphalt of the same composition (constant σ, 10°C, 50 Hz) with mean voids of 4.5, 7.0 and 9.0%. Their test results ($\varepsilon - n$ at failure graph) show for example that with 4.5% voids the number of load repetitions (n) to failure is 10^6 with $\varepsilon = 200 \times 10^{-6}$; this number is reduced to $3 \cdot 10^5$ with 9% voids. A 4.5% increase in voids thus reduces the service life on average to one third of its initial value. There is also a reduction in rigidity (approx. 3% – relative – per percent voids) so that the loss of service life for an identical stress level ($\sigma - n$ to failure graph) will be even greater.

Monismith and co-workers [14] carried out large scale fatigue tests in which, as in the research referred to above, the voids varied on the basis of compaction differences, in other words the degree of compaction of identical asphalt varied.
This research was conducted with two mixes of different composition: a BS 594 hot rolled asphalt with 30% chippings and 7.9% bitumen 40/50, and two Californian asphaltic concrete mixes with on average 55% chippings and approximately 6% bitumen (85/100). The Wöhler curves (relationship between strain at failure and number of load repetitions at which failure occurs, in the general form $\varepsilon = A \cdot n^{-b}$) for the two mixes show considerable differences, in particular a substantially higher

strain at failure and a flatter curve (higher exponent b) for BS 594 asphalt. This better fatigue characteristic was also reflected in behaviour with increasing voids; a 10% increase in voids for BS 594 asphalt and 4-6% increase for Californian asphaltic concrete caused a reduction in service life with a factor of 10. Since the voids content of the latter mix was distributed over larger voids, it was concluded that this voids distribution is the cause of the difference.

Rigidity is reduced as follows by the increase in voids: in the case of BS 594 asphalt *with 5% voids by a factor of 2/3 and 11% voids by a factor of 1/3; in the case of Cali-*fornian asphalt for a 5% increase in voids, on average by a factor of $\frac{1}{2}$.
The reduction in service life cannot, however, be fully explained by this loss of rigidity.

In recent years, Pell and Taylor [15] have conducted extensive series of fatigue tests and established a complete $\varepsilon - n$ graph for voids varying in 1% stages between 0 and 10%. The results relate to BS 594 roadbase asphalt, i.e. discontinuously graded asphaltic concrete with 60% porphyry chippings, round sand or crushed porphyry sand, limestone or sand filler and 6% bitumen 40/50, investigated with constant σ(1.2 MN/m^2), at 20°C and 20Hz.

The description of the research shows, however, that these results were not obtained by intentional variation of the degree of compaction but that they are primarily due to differences in mix composition as the origin of the varying voids (i.e. variations in bitumen and filler contents since other variations were not significant). The results show that an increase in voids from 0 to 10% gives a reduction in service life by a factor of 1/20 and that a 1% voids increase signifies a multiplication factor of slightly less than 3/4 for the service life (reduction of about 25% in service life).

J. M. Kirk [16] also performed a number of fatigue test (constant ε, 50 Hz) mainly with sand asphalt (e.g. based on crushed granite sand, limestone filler and 80/100 bitumen) but also with a number of very fine asphaltic concrete mixes with variable voids (1-18%). For these mixes, he reports the strain at failure at 10^6 load repetitions: for other mixes with high voids the strain at failure is relatively low, but these mixes also have a lower bitumen content to which the difference is attributed. Three mixes with the same bitumen content but differing by 6% in voids, show practically the same value for ε at $n = 10^6$; Kirk accordingly concludes that voids do not influence the fatigue strain.
The conclusion reached by Kirk differs completely from the results of the other research workers but is reported here for completeness. General considerations of material engineering also tend to throw doubt on Kirk's result.

The other research workers referred to above accordingly show a varying influence of the increase in voids on fatigue life. To sum up, the results in respect of a 10%

increase in voids due entirely to variations in the degree of compaction, show a reduction factor for service life of

$\frac{1}{16}$ (Saal and Pell, sand asphalt)
$\frac{1}{10}$ (Bazin and Saunier, sand asphalt)
$\frac{1}{100}$ (Epps and Monismith, Californian asphaltic concrete), and
$\frac{1}{10}$ (Epps and Monismith, BS 594 hot rolled asphalt)

and primarily due to mix variations

$\frac{1}{20}$ (Pell and Taylor, asphaltic concrete).

In practice, however, the variation in voids is a combination of variation in the degree of compaction and mix; in some cases one of these two factors predominates.

Having regard to the fact that the asphaltic concrete used in the Netherlands is more closely related to American 'Marshall asphalt' than to English 'hot rolled asphalt' and that its composition, under the influence of various other factors, is (unfortunately) increasingly tending to resemble that of Marshall asphalt, the assumption of a reduction in service life of 25% (relative) per volume per cent increase in voids would seem to be rather on the low side (factor 1/18 for 10% voids).

It is self-evident that the differences between the results obtained may be attributable to the widely varying test methods used by the different authors, but this aspect has not been studied in more detail.

A number of further observations are called for on the use made below of the fatigue life determined in laboratory tests of asphalt. It must not be assumed that the laboratory fatigue strength or strain is the decisive value in practice but only that the relative reduction determined in the laboratory will also be reflected in practice; furthermore the road design should be considered to be based on a fatigue strength which will be just adequate at the end of the service life of the road. It is accordingly assumed that the strain of the asphalt is in practice the decisive criterion for service life and that reduction in service life under practical conditions will be proportional to the laboratory situation.

This assumption will not always hold good, e.g. because during construction in accordance with the design data another criterion, soil pressure which will be dealt with later, may be the principal determinant, or because the location of the asphalt layer is such that it is subject to less strain. It must, however, be noted that, partly because of the reduced strain at failure of the asphalt, this characteristic which is contrary to the design data may become the decisive factor (and indeed often does as is shown by Brouwers [11]), while in a 'protected' asphalt layer the strain which occurs may increase with the passage of time so that this weakness criterion again applies. In the event of failure of the underlying layer as a result of an effective reduction in layer thickness even higher requirements may be placed on this material.

The second assumption can be allowed for in practice with rational design methods, in which the strain at failure calculated through laboratory fatigue tests is taken as the failure criterion.

So as not to make these calculations excessively complicated, it is assumed that the rigidity relationship of the total asphalt pavement (constructed out of several different layers) to the roadbase or subgrade is not changed; allowance is thus only made for changes in the failure criterion (determinant strain). This of course means that a further consequence, a change in the strain of the pavement occuring under load, is disregarded.
If this factor is to be taken into account, the structure will have to be the subject of further calculation, e.g. with the aid of the BISTRO programme.
A simple calculation is given here on the basis of which it can be seen that the strain which occurs with increased voids falls further: working from the calculation method described by Burmister (Fox, Acum and Fox), and Jeuffroy, Brouwers [17] concluded that the following expression is reasonably accurate:

$\log\frac{\sigma_r}{\sigma_o} = A \log\frac{E_2}{E_s} + B$, with which parallel lines are found for different values of $\frac{E_1}{E_2}$.

Here σ_r is the radial strain at the bottom of the asphalt layer
σ_o is the contact pressure
E_1, E_2 and E_s are the respective moduli of elasticity of asphalt, roadbase and subgrade (applicable to 2 and 3 layer systems with $\gamma = 0.5$).
The following expression [18] is a good approximation for the 2 layer system:

$$\frac{\sigma_r}{\sigma_o} = 1.8\left(\frac{a}{h}\right)^{1.56}\left(\frac{E}{E_s}\right)^{0.15}$$

applicable for Jeuffroy and Burmister if $E/E_s > 20$,
where h is the layer thickness
a is the radius of the contact surface
E is the modulus of elasticity of the pavement (rigidity)
E_s is the modulus of elasticity of the subgrade.

If the modulus of elasticity of the asphalt pavement is halved (according to Monismith this is the consequence of 5% higher voids in Californian asphaltic concrete), the tensile stress encountered at the bottom of the asphalt layer is only reduced by 10%. As a consequence of the halving of the modulus of elasticity, the strain which occurs will clearly be substantially greater. In disregarding this factor one is therefore postulating a far more favourable situation than exists in reality.
On the basis of the assumed 25% (relative) reduction in service life for each 1 per cent higher voids content, the financial consequences for the road authority and contractor must now be calculated.

The method of calculation is similar to that used for the layer thickness. This is done on the basis of the average situation for bitumen-bound gravel obtaining in the past, i.e. the 'average' pavement with mean voids of 5.9% and a standard deviation (s) of 1.8% is taken as standard and assumed to have a service life of 20 years. A 'cost-benefit' calculation is now effected for four different situations with higher average voids; on the one hand the financial compensation for the resulting reduction in service life is calculated, while on the other the penalty is determined with reference to the 'equivalent' statistical penalty provisions (see Table 9). The compensation is calculated in the same way as for layer thickness (with the costs of the asphalt layer set at f4 per m^2). It is assumed that the true standard deviation (σ) is and remains identical to the 'historical' mean value of 1.8%. Similar calculations based on different standard deviations are of course also possible but have not been carried out here. Consequently the calculation is not as complete as for the layer thickness, where allowance was also made for the consequences of increased maintenance due to dispersion in the layer thickness.

It is thus apparent that the proceeds of the penalty represent on average only 15% of the calculated costs to the road authority.

Table 9 Consequences of reduced permissible strain due to higher voids content in bitumen-bound gravel

cost			proceeds of penalty		
mean voids (vol. %)	mean life (years)	compensatory asphalt layer at end of service life, % value asphalt layer	(R = 9,5%) (s = 1,8%) $\frac{R-\mu}{\sigma}$	%defectives B	penalty as % value of asphalt layer (K = 0.38)
5.9	20	zero	$\frac{9.5-5.9}{1.8} = 2.00$	2.3	zero
6.9	17	$\left\{\frac{(1.08^3-1)8}{1.08^{20}} + \frac{1}{1.07^{17}}\right\}\frac{100}{4} = 18$	$\frac{9.5-6.9}{1.8} = 1.44$	7.5	2.3
7.9	$14\frac{1}{4}$	$\left\{\frac{(1.08^6-1)8}{1.08^{20}} + \frac{2}{1.08^{14}}\right\}\frac{100}{4} = 42$	$\frac{9.5-7.9}{1.8} = 0.89$	18.7	5.6
8.9	$11\frac{3}{4}$	$\left\{\frac{(1.08^8-1)8}{1.08^{20}} + \frac{3}{1.08^{12}}\right\}\frac{100}{4} = 66$	$\frac{9.5-8.9}{1.8} = 0.33$	37.0	11.1
9.5	10	$\left\{\frac{(1.08^{10}-1)8}{1.08^{20}} + \frac{4}{1.08^{10}}\right\}\frac{100}{4} = 96$ or $\frac{8}{1.08^{10}} \cdot \frac{100}{4} = 93$ based on add. treatment excl. compensation for loss of use	$\frac{9.5-9.5}{1.8} = 0$	50.0	15.0

These calculations show the same overall pattern for open-textured and dense asphaltic concrete (see Table 10).
It should be noted that the values for μ and σ chosen for open-textured and dense asphaltic concrete are the average values for mean voids and mean standard deviation found by analysis of all projects on which penalties have been imposed hitherto [8].
The value chosen for the limit value R is also based on the proposed new statistical procedure for the determination of penalties [8].
The limit value used at present for penalty calculations is $R = 9.5\%$ for open-textured asphaltic concrete and $R = 6.5\%$ for dense asphaltic concrete.
Where these limit values are used, the results shown in brackets should be obtained. In that case there is a much greater discrepancy between the calculated penalty and the costs calculated on the basis of the reduction in service life for open-textured asphaltic concrete. In the case of dense asphaltic concrete the difference is less marked, corresponding to a factor of only $4\frac{1}{2}$ instead of 6.
When open-textured asphaltic concrete is applied as basecourse on a sufficiently rigid road base, a reduction in the permissible strain may be less applicable; however the consequences of the reduction in rigidity will still apply.
In the foregoing, only the consequences of a reduction in the strain at failure of asphalt as a result of an increase in voids have been considered and the increase in the strain attributable to a reduction of rigidity has been disregarded.

Table 10 Penalties for open-textured and dense asphaltic concrete due to higher voids content

open-textured asphaltic concrete		$R = 8.5\%$ (9.5%) $s = 1.9\%$		dense asphaltic concrete		$R = 7.0\%$ (6.5%) $s = 1.65\%$	
mean voids (vol. %)	$\frac{R-\mu}{\sigma}$	percentage defectives B	penalty as % value of asphalt layer $K = 0.3B$	mean voids (vol. %)	$\frac{R-\mu}{\sigma}$	excess percentage defectives B	penalty as % value of asphalt layer $K = 0.3B$
4.7 mean standard	2.00 (2.53)	2.3 (0.57)	zero	3.7 mean standard	2.00 (1.70)	2.3 (4.5)	zero
5.7	1.47 (2.00)	7.1 (2.3)	2.1 (zero)	4.7	1.39 (1.09)	8.2 (13.8)	2.5 (4.1)
6.7	0.95 (1.47)	17.1 (7.1)	5.1 (2.1)	5.7	0.79 (0.49)	21.5 (31.2)	6.5 (9.4)
7.7	0.42 (0.95)	33.7 (17.1)	10.1 (5.1)	6.7	0.18 (0.12)	42.9 (54.8)	12.9 (16.4)
8.5	0 (0.53)	50 (29.8)	15.0 (8.9)	7.0	0 (0.30)	50 (61.8)	15.0 (18.5)

However, a further consequence of this loss of rigidity which can easily be determined in numerical terms must now be considered, i.e. the increase in the occurring soil pressure.
As we have already seen, according to Epps and Monismith the loss of rigidity for two widely different asphalt mixes is represented by a factor of 1/2 and 2/3 respectively for 5% voids, corresponding to a reduction of 8 and 13% respectively per percent voids.
11% is taken as the mean value here, i.e. a reduction factor of 0.89.
The relationship between the soil pressure σ_v (directly below the pavement in the centre of the load) can generally be indicated as follows according to Brouwers [17] for the 2 and 3 layer system used by Burmister and Jeuffroy:

$\log \frac{\sigma_{v2}}{\sigma_o} = -A \log \frac{E_2}{E_3} - B$ in which A is practically always equal to 2/3, and

$$B = \mathrm{f}\left(\frac{a}{h_1}, \frac{a}{h_2}, \frac{E_1}{E_2}\right).$$

The following expression is arrived at for the 2 layer system [18]:

$\sigma_v = 2\sigma_o \left(\frac{a}{h}\right)^2 \left(\frac{E_s}{E}\right)^{2/3}$ applicable to Burmister and Jeuffroy's system where $\frac{E}{E_s} \geqslant 40$.

In the complete asphalt structure, considered for greater simplicity as a 2 layer system, σ_v is thus inversely proportional to the rigidity of the asphalt to a power of 2/3. This means that a reduction in E by a factor of 0.89 results in a reduction of σ_v by a factor of 1.08.
On the basis of the AASHO test results a relationship of the type $\sigma_v = A\, n^{-5}$ has been established between the service life expressed in axle passages of an equivalent axle (n) and the soil pressure (σ_v); this expression has been used as the basis for the Shell design method [19].
This means that an increase of σ_v by a factor of 1.08 leads to a reduction of n by a factor of 0.68, i.e. approx. 2/3.
This result is even more unfavourable than the factor of 3/4 calculated for the reduction in strain at failure.
These factors obviously have consequences if the soil pressure is or becomes decisive for the pavement as is assumed in the Shell design method, e.g. for thick asphalt structures (of good quality in terms of strain at failure).
Comparative calculations ('cost-benefit analyses') have been made on the basis of this 2/3 factor which reduces the service life (Table 11).
In drawing the comparison between the costs of compensating asphalt and the yield of the penalty, it has been assumed here (for greater simplicity) that the entire struc-

Table 11 Consequences of reduced rigidity of the entire structure due to higher voids content

increase in voids above standard, all layers, %	mean service *life in* years	cost of compensatory asphalt layer *at end of service life*, % value of total pavement	proceeds of penalty as % value of total pavement, mean of 5 layers (see Tables 6 and 7)
0	20	zero	zero
1	16	$\left\{\frac{(1.08^4-1)8}{1.08^{20}}+\frac{2}{1.08^{16}}\right\}\frac{100}{20}=6$	$\frac{3\times 2.3+2.1+2.5}{5}=2.3$
2	$12\frac{1}{4}$	$\left\{\frac{(1.08^8-1)8}{1.08^{20}}+\frac{3}{1.08^{12}}\right\}\frac{100}{20}=13$	$\frac{3\times 5.6+5.1+6.5}{5}=5.7$
3	9	$\left\{\frac{(1.08^{11}-1)8}{1.08^{20}}+\frac{4}{1.08^{9}}\right\}\frac{100}{20}=21$ or $\frac{8}{1.08^9}\cdot\frac{100}{4}=20$ based on additional treatment, excluding compensation for loss of use	$\frac{3\times 11.1+10.1+13.3}{5}=11.3$

ture (i.e. all the layers) will have a given higher voids content (as compared with the mean standard voids) so that the 'yield' and the costs are expressed as a percentage of the cost of the entire pavement.

In these calculations too there is a discrepancy between the compensation costs to be fixed on the basis of the deterioration of material characteristics and the yield of the penalty imposed under the relevant provisions.
Finally it should be noted that in practice it will generally not be possible to determine as such the possible loss of quality or damage resulting from the material-technical phenomena described here. It is also not really known to what extent a safety margin is included under practical conditions, as a result of which deviations from the mean (which is treated under the penalty provisions as a feasible aim) during the normal service life (which may in fact also be limited by other factors) will not lead to manifest increases in voids. Should this be so, it would still not disprove the value of the cost-compensation method used. The relationship between the voids and all the 'strength characteristics' of the material, and the interrelationship between the latter and the design thickness, make it reasonable to suppose that this particular property is of considerable importance to the behaviour pattern, especially and above all as regards the durability (service life) of the structure.

It will perhaps not be superfluous to point out that in this study various other consequences of higher voids which also influence durability (mechanical deterioration and stripping of the binder under the influence of climate and traffic loads, especially in the case of the surface layers) have been entirely disregarded.

Bitumen content

The consequences of deviations in the asphaltic bitumen contents can be calculated in the same way as has been described in detail for the voids. Saal and Pell [12] and Jimenez and Gallaway [20] have already indicated the great influence of the asphaltic bitumen content on the service life in fatigue tests of asphalt. In their first fatigue tests, Saal and Pell already gained the impression that the permissible strain in the binder ε_B is the determining parameter. The strain in the mix is then $\varepsilon_m = B_v \cdot \varepsilon_B$, where B_v is the volumetric concentration of the bitumen in the mix. Jimenez and Gallaway found an optimum binder content at which the service life is greatest for a given asphalt mix.
The first finding has been confirmed by other research workers (naturally below the optimal binder content), e.g. by Kirk [16) ($8 \cdot 10^{-6}$ per % (V/V) bitumen at $n = 10^6$) and Epps and Monismith [14], while an optimum for a curve giving the service life as a function of the binder content was later also found by Pell [21], Pell and Taylor [15] and Epps and Monismith [14].

Pell's assumption that the relationship between n and ε_B is decisive, with $n = A \cdot \varepsilon_B^{-5}$, gives, in conjunction with $\varepsilon_M = B_v \cdot \varepsilon_B$: $n = A \cdot B_v^{-5} \cdot \varepsilon_M^{-5}$ or $\varepsilon_M = A' \cdot B_v \cdot n^{-\frac{1}{5}}$. This means that for n = constant the permissible strain in the mix is proportional to the volumetric concentration of the bitumen, while for an identical strain level the permissible number of load repetitions (the service life) is inversely proportional to B_v^5, i.e.

$$n = n_o \left\{ \frac{B_v}{B_{v_o}} \right\}^5$$

This means for example that in a comparison between an asphalt with 7.0% (m/m) and an asphalt with 6.3% bitumen (10% less) – with (given 0% voids and a mineral density of 2.65) $B_v = 0.15$ and 0.14 respectively (ratio of 0.93) – the service life in the case of 6.3% binder will be 0.7 times the life with 7.0% bitumen.

This overall approach to the problem has in fact been superseded by the practical determination of the service life of a number of asphalt mixes as a function of the binder content.
Pell's study [21] related in the first instance to a BS 594 roadbase asphalt (discontinuously graded) with 60% porphyry chippings and sand to which, in accordance with the relevant specifications, no filler is added, and 4-12,5% bitumen 40/50.
Under Pell's test conditions ($\sigma_{max} = 1/2$ N/mm², 10°C, 20 Hz) a definite maximum service life of $n = 1.5 \cdot 10^5$ was found for 7.7% bitumen. His graph for $n = f$ (bitumen %) shows that $n \geqslant 10^5$ between 6.5 and 10% bitumen, while n falls much more rapidly with lower than with higher binder contents ($n = 3.10^3$ for 4% bitumen).

The voids varied between 0 and 9.8% and corresponded to 0.9% in the case of the optimum mix. This study was followed by a further study in which for mixes with the desired binder content of 6%, according to BS 594, the filler content (limestone) was varied between 0 and 17%. Here again a definite optimum service life was found for 9.7% filler; the original value $n = 8.10^4$ (without filler) was increased in this case to $2.5 \cdot 10^6$. The voids ranged from 0 to 4.5% (0 in the optimum case). Finally Pell and Taylor [8] kept a constant filler content (10.3%) but varied the binder content between 3.5 and 10.5%; it was then found that 6.4% bitumen 40/50 gave an optimal service life ($2.5 \cdot 10^6$) but this fell off quickly, expecially with lower binder contents. Even at the limits of the BS 594 specification, i.e. 5.9.% min. and 7.1% max., a considerable reduction is found, i.e. to $4 \cdot 10^4$ and $1.05 \cdot 10^5$. The voids varied between 0 and 5.4%.
In Pell's first study, the optimum mix with 8.3% bitumen also showed a maximum rigidity of $7.5 \cdot 10^3$ N/mm^2 (probably because with a lower bitumen content the increase is counteracted by the higher voids content; in Pell and Taylor's series a mix with a lower binder content had a higher rigidity than the optimum mix ($11 \cdot 10^3$ MN/m^2). The decisive strain level in these tests was 100 to $150 \cdot 10^{-6}$. A further striking feature is that the optimum binder content is 6.3% with 10.3% filler, but substantially greater, i.e. 8.3%, without filler.

The fact that specific mix characteristics may influence the optimum bitumen content and the corresponding service life was already observed by Jimenez and Gallaway, who found in particular that a rougher, more absorbent aggregate had a higher optimum bitumen content and a longer service life. Pell and Taylor also compared a sand filler with limestone which gave lower results.

Epps and Monismith also studied the influence of the bitumen content for a number of mixes. This was done first with two asphaltic concrete mixes on the basis of two types of basalt aggregate (with approx. 50% chippings, max. 12 mm, 6% filler and 5.3-8.7% bitumen 60/70). Under his test conditions ($\sigma_{max} = 1.1$ MN/m^2, 20°C, 0.1 s and a strain level of approx. 500 μm), Monismith found a maximum $n = 1.9 \cdot 10^4$ at 6.7% bitumen. As in Pell's study, n drops very rapidly with lower asphaltic bitumen content and less rapidly with a higher content (still 10^4 at 8.5%). A maximum rigidity is again found ($2.7 \cdot 10^3$ MN/m^2) at the optimum binder content. The voids in this case were 5.2% and varied between 1.6 and 8.8% in other instances. The optimum binder content for maximum service life at 6.7% was 0.8% higher than on the basis of the Californian (stability) tests (cohesiometer) while the service life is then roughly three times greater.
Analysis of the results obtained by Pell and Taylor for the mix with 10% limestone filler (far more comparable with our own asphalt mixes than the mix without filler) shows that the life of $2.47 \cdot 10^6$ at 6.4% bitumen content is reduced to $4.43 \cdot 10^5$ with 4.5% bitumen, and to $1.05 \cdot 10^4$ with 3.5% bitumen. However, with these mixes

the voids rise from 0.4 to 2.8 or 5.4% so that (on the basis of the data indicated in the chapter on voids) a halving of the service life can be shown for each successive bitumen content. The reduction factor resulting from the respective lowering of the bitumen content by 1.9 and 1.0% is then 0.36 and 0.05 respectively which corresponds to a reduction factor of 0.90 and 0.55 respectively per 0.2% bitumen.
This reduction therefore increases considerably with a very low bitumen content (departing from the general formulation based on B_v).
Epps and Monismith describe a reduction from $n = 19.1 \cdot 10^3$ for 6.7% bitumen and 5.2% voids to $n = 6.6 \cdot 10^3$ at 6.2% bitumen and 6.2% voids. After correction for the considerable drop in n found by them at higher voids content (factor 0.63 per % voids) we obtain a reduction factor of 0.55 for 0.5% bitumen or 0.80 for 0.2% bitumen.
For an even lower binder content, Epps and Monismith describe a less far-reaching reduction in n. Further analysis of their results shows, however, that this reduction can also be attributed to the further increase in voids content.
It is therefore considered reasonable in our further discussion to take a mean reduction factor for service life of 0.85 per 0.2% bitumen applicable to binder contents which are not reduced too far.
For raised bitumen contents the reduction factor is lower, i.e. a factor of approximately 0.6 per % bitumen.
The results of calculations to which the same assumptions apply as for voids are shown in Table 12.

The calculation is effected for coarse, dense asphaltic concrete and, in addition to the assumed mean standard deviation of 0.29% (identical to the historical average for all penalty contracts), a calculation has also been made of the penalty for an unnecessarily high standard deviation of 0.35%.
As in the case of voids we did not investigate the effect of higher standard deviations on the increase in maintenance costs.
Similar calculations can be made for other asphalt layers. The results obtained here show that the yield of the penalty is always substantially lower than the damage actually incurred.
For higher bitumen contents the results will be less unfavourable, but it must be remembered that with high binder contents other types of damage which are difficult to quantify are likely to occur.

If the mean bitumen content is reduced a quantity of binder the cost of which can be calculated will not be supplied. In the examples quoted with 6.3, 6.1 and 5.85% bitumen respectively, the value of this non-delivery is not less than 0.6%, 1.2% and 1.9% (overall) respectively of the value of the asphalt (on the basis of prices obtaining in mid-1972). This percentage must be deducted from the proceeds of the penalty if the comparison is to be effected in the same manner as in respect of the other characteristics.

Table 12 Coarse dense asphaltic concrete: consequences of reduced bitumen content

mean bitumen content %	mean service life (years)	compensatory asphalt layer at end of service life as % value of asphalt layer	proceeds of penalty $R = 5.85\%$ $\frac{R-\mu}{\sigma}$	$s = 0.28\%$ 0.35% per cent defectives B	penalty as % value asphalt layer ($K = 0.3\,B$)
6.5	20	zero	$\frac{6.5 - 5.85}{0.29} = 2.24$	1.25	none
			$\frac{6.5 - 5.85}{0.35} = 1.86$	3.1	none
6.3	$18\frac{1}{4}$ (18)	$\left\{\frac{(1.08^2 - 1)8}{1.08^{20}} + \frac{1}{1.08^{18}}\right\}\frac{100}{4} = 13$	$\frac{6.3 - 5.85}{0.29} = 1.55$	6.1	none
			$\frac{6.3 - 5.85}{0.35} = 1.29$	9.9	3.0
6.1	$16\frac{3}{4}$ (17)	$\left\{\frac{(1.08^3 - 1)8}{1.08^{20}} + \frac{1}{1.08^{17}}\right\}\frac{100}{4} = 18$	$\frac{6.1 - 5.85}{0.29} = 0.86$	19.5	5.9
			$\frac{6.1 - 5.85}{0.35} = 0.71$	23.9	7.2
5.85	15	$\left\{\frac{(1.08^5 - 1)8}{1.08^{20}} + \frac{2}{1.08^{15}}\right\}\frac{100}{4} = 36$	$\frac{5.85 - 5.85}{\sigma} = 0$	50	15.0

Conclusion

The penalties calculated with the aid of the new statistical method of evaluation amount on average to 30, 15 and 30% respectively of the necessary compensation costs which can be calculated on the basis of the reduction in the service life of an asphalt pavement as a consequence of certain shortcomings in respect of layer thickness, voids and bitumen content.

5 Summary

A uniform, effective and equitable system of quality control for pavements has been applied to road-building projects in the Netherlands since 1968. The system has been developed in consultation between the public authorities and the contractors' organizations.

The system is characterized by a distinction between regular, daily production control by the contractor and limited retrospective acceptance control by the public authority based on random samples.

Under the existing non-statistical system, samples (one per 2000 m^2) are examined to determine the layer thicknesses, sand cement strength and density and bitumen content of the asphalt.

When the quality does not meet the specified values, financial penalties are applied the level of which depends on the extent of the deviation from the set standards. In the light of experience with over 300 major projects, the system has now been developed into an almost entirely statistical method of control which is ready for practical introduction.

Under this statistical system, samples are examined for road surface areas up to a maximum of 200,000 m^2 ($n = 20$ or 40). Penalties are imposed if $\frac{|R - \bar{x}|}{s} < Q$,

where R is the quality limit, $\bar{x}$ the mean value, s the standard deviation and Q the quality index ($Q = 1.6$ or 1.4).

Penalties are fixed by the formula $K = 0.3\,B - C$, where K is the penalty as a percentage of the pavement costs, B the percentage of unsatisfactory work and $C = 1.0$ or 2.0

Control of the surface characteristics relates to skid-resistance and evenness measured at random points on 30% of the pavement surface.

Penalties are imposed if the measurement results do not meet the criteria. If the results fall below certain safety limits the surfaces must be repaired at the contractor's cost.

A theoretical study of the shortening of the service life of a road pavement due to deviations in the thickness, density and bitumen content of asphalt layers, leads to the conclusion that the penalties imposed under the statistical quality control system are substantially lower (on average three times lower) than the calculated costs theoretically necessary as compensation for the reduction in value due to the shorter service life. Calculations of this kind are set out in the final chapter. The reduction in the service life of the pavement as a consequence of specific shortfalls on layer thickness, excessive voids and too low bitumen content is calculated. The consequences of changes in the service life pattern in relation to reconstruction and maintenance,

in addition to the costs of the resulting necessary additional work in comparison with the specified work structure, are also calculated.

Finally, the amount needed after laying the pavement to cover the costs calculated in this way is also determined to give the necessary compensation costs. These amounts are compared with the proceeds of the penalties imposed in such cases.

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