BULETINUL INSTITUTULUI POLITEHNIC DIN IAŞI
Tomul LI (LV), Fasc. 1–2, 2005
CONSTRUCŢII. ARHITECTURĂ

PROBABILISTIC CORROSION SERVICE LIFE MODEL OF CONCRETE BRIDGE EXPOSED TO CHLORIDE DEICER SALTS

BY

CRISTIAN-CLAUDIU COMISU

A model to determine the time till first repair and subsequent rehabilitation of concrete bridge decks exposed to chloride deicer salts that recognizes and incorporates the statistical nature of factors affecting the corrosion process, consists of the following serial phases: diffusion to the depth of the steel that would precipitate first maintenance actions; corrosion of the steel at the first maintenance depth until cracking and spalling occurs; continuous spalling until a damage level is reached which is defined as the end of functional service life for the deck of concrete bridge.

The model expands on an existing deterministic model using statistical computing techniques, including resampling techniques such as the parametric and simple bootstrap. Emphasis was placed on the diffusion portion of the diffusion–cracking model, but advances can be readily included for the time for corrosion deterioration after corrosion initiation. The diffusion phase is described by Fick's law and the boundary conditions define the solution form. The time till cracking model depends upon some factors as concrete strength properties, cover depth and corrosion rate. Corrosion rate has a significant influence on the time to cracking which typically occurs in three to seven years after initiation.

Service life model that predict the time to first repair and rehabilitation of concrete bridge decks provide a useful tool for comparing the effectiveness measures and provide bridge engineers with a useful planning tool.

A service life model for the corrosion of concrete bridge deck in chloride laden environments is proposed. The aim of this paper is to enhance the awareness among the engineering community to use the model to determine the time till first repair and subsequent rehabilitation of concrete bridge decks exposed to chloride deicer salts in Romania.

1. Introduction

According to the Federal Highway Administration, approximately 30% of the nation's bridges are either structurally deficient or functionally obsolete. For many of these bridges, the concrete decks will need to be rehabilitated before other components of the bridge, and *prior* as the previous predicted design service life of 50 years. In Virginia approximately 28% of the bridges on Interstates and major highways are considered structurally deficient or functionally obsolete. While corrosion of the reinforcing steel was not the sole cause of all structural deficiencies, it was a significant contributor and has therefore become a matter of major concern.

The magnitude of this corrosion problem in the transportation infrastructure has increased significantly in the last three decades and is likely to keep increasing. Even though the cost of maintaining bridge decks is becoming prohibitively expensive, the benefits provided by deicing salts are too great, however, that it's use is not likely to decrease in the future. In fact, the use of road deicing salts, which are extremely corrosive due to the disruptive effects of its chloride ions on protective films on metals, has actually increased in the first half of the 1990s, after a leveling off during the 1980s. The main cause is the "bare pavement" policy which was adopted by highway agencies to promote safe driving during winter weather conditions. The bare pavement policy was achieved through an increase in the use of chloride deicing salts.

Thus, while embedded steel in concrete is normally passive and corrosion rate accordingly low, accumulation of chlorides at the steel depth in a critical amount compromises the protective film and, in the conjoint presence of moisture and oxigen, induces active corrosion. The resultant solid corrosion products accumulate in the concrete pore structure immediate to the steel–concrete interface and induce tensile stresses in the concrete. Because concrete is relatively weak in tension, cracking and spalling ultimately follow. Consequently, a large number of corrosion protection measures have been used to delay the deterioration caused by chloride-induced reinforcement corrosion [1].

2. Methods of Providing Corrosion Protection

The expected service life of a newly constructed bridge is typically 75 years and up to 120 years for stainless steel rebar construction. Many rehabilitation methodologies designed to extend the service life of bridges that have deteriorated due to corrosion of the reinforcing steel have been developed and put into practice within the past 25 years. The basic methods of providing corrosion protection for steel in concrete are: preventing or slowing the ingress of chlorides through the concrete, altering the electrochemical nature of the system, and protecting the exposed surface of the steel reinforcement. Although each of these methods have been shown to be successful, continuing developments are necessary to improve effectiveness and increase the life extension provided by these methods.

Methods that prevent or slow the ingress of chlorides include low permeability concretes with low water cement ratios, concretes with supplementary cementitious materials (such as fly ash, micro-silica, or ground granulated blast-furnace slag), increased clear cover depth, polymer overlays, and deck sealers. These methods seek to prevent the time for chlorides to reach the reinforcing steel, and are often thought of as the first line of defense against corrosion.

Cathodic protection of the reinforcing steel represents a method that alters the electrochemical nature of the system. Cathodic protection involves the use of a sacrificial anode that reverses the flow of electrons, and forces the steel to become the cathode, or non-corroding, portion of the corrosion cell. Cathodic protection requires the steel reinforcement to be electrically interconnected and requires maintenance to ensure that the sacrificial anode is plentiful.

Protection methods that protect the exposed surface of the steel include the use of galvanized reinforcement and epoxy coatings. The use of epoxy coated reinforcement (ECR) is widespread in the United States. However, research suggests that the epoxy coating on ECR will debond in 4...15 years. For most bridge decks built today, the chlorides will not reach the surface of the steel until after the epoxy coating has debonded. Therefore, the effectiveness of ECR as a corrosion protection measure is in question [1], [2].

3. Service Life Model

While many protection measures have been used successfully, they need to be evaluated on a life cycle cost basis. Service life models have been used for many years. Typically the models are deterministic and use mean values for the input parameters. However, in recent years, there has been a trend towards incorporating the statistical nature of the input parameters into service life models.

Because decisions on bridge rehabilitation are typically decided based largely on the condition of the bridge deck, this study will use the simplified Cady-Weyers model for bridge decks which deteriorate by chloride induced corrosion of the reinforcement.

A better service life model would assist planners in two ways. First, the remaining time till first repair and subsequent rehabilitation could be estimated with greater accuracy for a given bridge or set of bridges. Second, the effectiveness of the various protection methods could be compared and evaluated.

The service lifemodel for the chloride induced corrosion of reinforcing steel in concrete involves two time periods and is presented in Fig. 1. The two time periods in this model are a diffusion period where chlorides diffuse through the concrete till

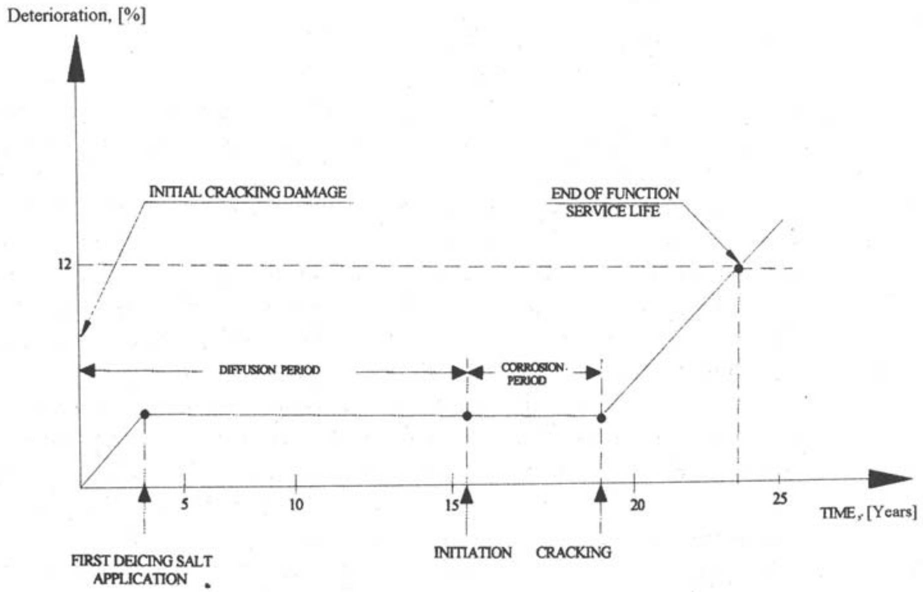

Fig. 1.– Chloride corrosion deterioration process for concrete bridge deck.

a concentration equal to the initiation concentration at the depth of the reinforcing steel and a corrosion period where the steel reinforcement corrodes and the deck deteriorates to the end of function service life (EFLS).

The equation for service life takes the following form:

Service life = Diffusion time + Corrosion time to EFSL

In Virginia the EFLS is typically defined as 12% deterioration of the worst span lane of the deck. In addition, the time to first repair is typically defined as 2.5% deterioration of that span lane. The percentage of deck deteriorated corresponds to the percentile depth of the steel reinforcement, which has been shown to be normally distributed for bridge decks. Service life models that predict the time till first repair and rehabilitation of concrete bridge decks provide a useful tool for comparing the effectiveness of corrosion protection measures and provide bridge engineers with a useful planning tool [2], [3].

An apparent diffusion process can be used to model the time for chloride to reach and initiate corrosion at first repair and rehabilitation reinforcing steel depths. When solved for the condition of constant surface chloride and a one-dimensional infinite depth, Fick's second law takes the following form:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left(D_c \frac{\partial C}{\partial x}\right),$$

where: $\partial C/\partial t$ is the variation in concentration of the diffusing species at a location of interest with time, D_c – apparent diffusion coefficient, C – concentration of the diffusing species, x – distance, and its solution, assuming that D is independent w.r.t. C,

$$C(x,t) = C_0\left(1 - \operatorname{erf}\frac{x}{2\sqrt{D_c t}}\right), \tag{1}$$

where: $C(x,t)$ is the chloride concentration as function of depth and time, C_0 – surface chloride concentration, t – time for diffusion, x – concrete cover depth, erf – statistical error function.

When $C(x,t)$ is set equal to the chloride corrosion initiation concentration (C_{crit}), and Eq. (1) is solved w.r.t. t, the time for diffusion of chloride ions to the chloride corrosion initiation concentration can be determined. However, for a given bridge deck, the values of $C(x,t)$, C_0, D_c and x are random variables, each with their own statistical distributions, means, and variances. A solution of Eq. (1) w.r.t. time for diffusion should include the probabilistic nature of the input variables.

The time for corrosion damage till the end of functional service life is likely a random variable as well and depends on the corrosion rate, concrete cover depth, reinforcing steel bar spacing, and size. However, little is known about the possible distribution of the time for corrosion damage; therefore, in this research, the value is taken as a point estimate [2], [4].

Because the input parameters in the service life model are random variables, a statistical tool is required to determine the service life. There are three types of probabilistic models in use today, and they include regression models, Markov chain models and statistical computing techniques applied to existing models.

Regression models relate service life to a few important input parameters. For a given set of data, a best-fit equation is generated that most accurately reflects the service life predicted by the various input parameters. One model uses regression to predict the apparent diffusion coefficient used in a web based service life predictor. Regression models may provide accurate results for a given data set, but must be used with caution when they are applied to a new sample population. Therefore, regression models are not widely used for service life predictions of individual or populations of bridge decks.

Markov chain models are used to predict the service life of a network of bridges using condition ratings determined when bridges are visually inspected. First, the bridges are assigned an initial condition rating based on the visual inspection. Then, the probability of a bridge deteriorating from one rating to the next in one year, called the *transitional probability*, is determined. Finally, the number of bridges reaching a rating where rehabilitation is required is determined for a given number of years. Service life models based on visual ratings tend to be highly variable because different inspectors often assign a different rating to the same bridge. In addition, these models may not provide accurate results because of the difficulty in determining the transitional probabilities.

The most widely used probabilistic models apply statistical computing techniques, such as Monte Carlo simulation, to existing deterministic models. The use of Monte Carlo techniques has been significantly enhanced by modern computing power, since a large number of iterations must be performed to obtain sufficiently descriptive results. Several computer packages capable to perform these simulations are available today [1], [5].

4. Schematic of Basic Routine

Monte Carlo is a class of repeated sampling methods where a value is randomly sampled from theoretical distributions, for example $C(x,t)$, C_0, D_c and x. Then, a mathematical model is solved for the desired response. The input parameters used in a mathematical model representing the service life are random variables with known distributions. Using a statistical computing technique, the model is solved for the desired response, for example, the service life. The result is an estimate of the service life that takes into account the statistical distribution of the input parameters. The entire process is repeated a sufficient number of times to define a distribution of the response. In this case, the solution of Eq. (1) w.r.t. time for diffusion is added to the time for corrosion damage till the end of functional service life a sufficient number of times to define a distribution of the service life. A related resampling method, called *bootstrapping*, uses the same repetitive sampling procedure but uses data to define the parameters for the distributions or samples directly from the existing data. Two types of the bootstrap, the parametric and simple bootstrap, shall be used in this research.

For a given bridge or set of bridges, the input parameters for the simulation routine include field data for x, C_0, and D_c, the time till corrosion deterioration after initiation, the range of the expected chloride initiation concentration, and the number of iterations. The basic routine is presented in Fig. 2 and is identical for both the parametric and simple bootstrap, except when the parametric bootstrap data is generated from the assumed distributions with optimal parameter values estimated from the data. Both methods were considered to test how dependent the results were on distributional assumptions. The basic outline of the parametric bootstrap is described first, and then differences in the simple bootstrap are highlighted.

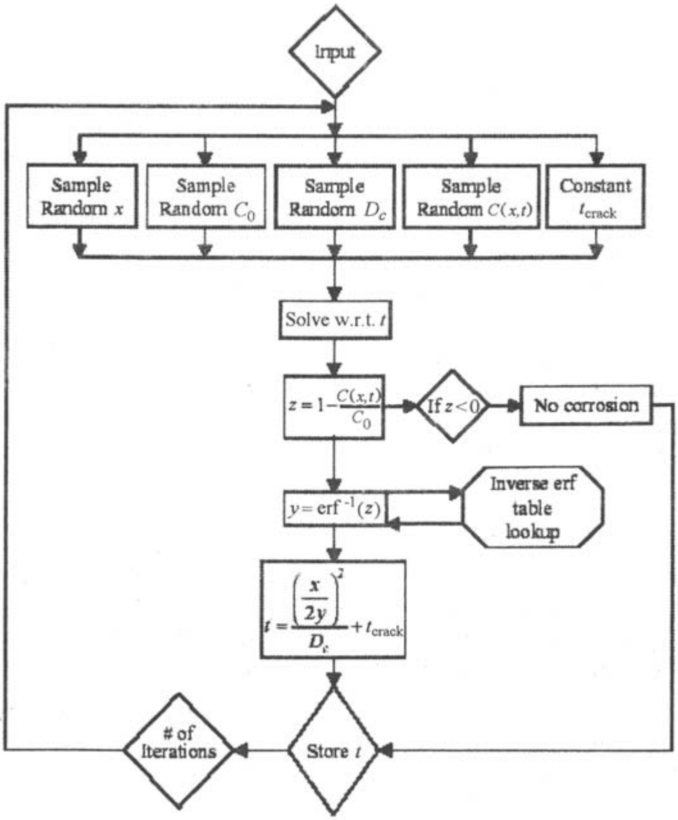

Fig. 2.– Schematic of basic routine.

For a given bridge deck, values of C_0 and D_c can be determined from chloride concentration profiles measured through the depth of the deck, and values of x can be measured using a rebar locator. It is best to measure the chloride contents directly over the reinforcing steel till at least partially or wholly account for the presence of the reinforcing steel. Here it is noted that Eq. (1) represents the solution for a one-dimensional analysis for infinite depth, whereas the presence of the reinforcing steel has been shown to significantly influence the rate of chloride increase at bar locations. The sample populations obtained from field data can then be used as the basis for the simulations. The distribution of the corrosion initiation concentration of chloride, $C(x,t)$, is unknown. A range of approximately 0.6...5.5 kg/m^3 has been commonly reported in the literature [1].

For bridge decks, the surface chloride concentration, C_0, is commonly taken as the concentration of chlorides located 12.7 mm below the deck surface. For the field drilled powdered and core samples, the first sample removed from the deck was centered at 12.7 mm below the surface, and therefore represents C_0. The surface chloride concentration was best described by a gamma distribution. It is important to note that, according to the model, if the value of C_0 is smaller than the chloride initiation concentration, corrosion will not take place on the bridge deck.

The apparent diffusion coefficient, D_c, is back calculated from each set of chloride concentration measurements obtained from the bridge decks. A minimum sum of square error procedure was used to back calculate D_c from the measured chloride profiles.

Using the measured C_0 value and one deeper chloride value, a trial D_c is back calculated from Eq. (1). Then, using the trial D_c, Eq. (1) is plotted against the actual chloride profile. The sum of square errors between the actual chloride profile and the chloride concentration predicted by the trial D_c is determined. Then, using an iterative procedure, the trial D_c is altered. A new sum of square error is calculated for each new trial D_c. Finally, the D_c value that predicts the chloride concentration with the minimum sum of square errors, when compared with the measured chloride profile, is taken as the best apparent diffusion coefficient. The same number of D_c and C_0 values is available for each bridge deck. The apparent diffusion coefficient was best described by a gamma distribution [2], [6].

5. Concentration of Chlorides Needed to Induce Corrosion and Time for Corrosion

It is clear that steel reinforcement in concrete will corrode in the presence of chlorides. However, the exact mechanism and concentration of chlorides needed to induce corrosion are not presently known. The initiation concentration may be a function of concrete mix proportions,cement type, C_3A content, blended materials, w/c ratio, temperature, relative humidity, steel surface conditions and source of chloride penetration. A value of 0.72 kg/m^3 is commonly used as an estimate of the minimum amount of chlorides needed to initiate corrosion.

A recent comprehensive review of the literature suggests that the initiation concentration should not be viewed as an absolute concentration beyond which corrosion of the reinforcement will take place. Based on experience, a range of 0.6...1.2 kg/m^3 has been suggested as a conservative estimate for use in the C a d y - W e y e r s service life model [7].

Work on field structures in the United States reported that the range of chloride initiation is between 0.59 and 5.1 kg/m^3. Similar work on field structures in the UK reported a range of initiation values between 0.7 and 5.25 kg/m^3. A slightly smaller range, 1.05...2.45 kg/m^3, was reported in a study of Danish bridges. A much higher range, 6.3...7.7 kg/m^3, was reported for Austrian bridges. A laboratory study in Japan, using simulated bridge substructure components, reported the chloride initiation concentration as a lognormal distribution with an average of 3.07 kg/m^3 and a standard deviation of 1.26 kg/m^3.

The influence of the epoxy coating on reinforcing steel on the chloride initiation concentration is currently under debate. Some have suggested that the initiation concentration should be larger for ECR than for bare bar. However, others note that inherent defects in the epoxy coating leave bare bar exposed to the chlorides, and therefore the initiation concentration will be the same for ECR and bare bars. Sagues estimated a range of initiation concentrations values for ECR that were of the same order as for bare bar.

Because defects in the coating expose bare reinforcement, it is reasonable that corrosion initiation takes place on ECR at the same concentration as bare reinforcement. Given the present state of the literature, it is appropriate to use a range of chloride initiation values between approximately 0.6 and 5 kg/m^3 for both bare reinforcement and ECR.

The generally accepted mechanism of chloride-induced corrosion is that chlorides break down the passive layer naturally formed on the surface of the steel in the normally high pH medium of concrete. Another proposed mechanism suggests that the naturally high resistivity of the concrete, which is expected to produce a low corrosion rate, is reduced in the presence of chlorides, and therefore the corrosion rate is increased. It has also been suggested that mineral scales are formed on the surface of the steel and protect the steel from corrosion.

In recent years, there has been debate regarding the influence of chloride binding on the initiation concentration. It has been suggested that only free chlorides are available for the corrosion process. However, current literature suggests that even chlorides chemically bound to the concrete during the diffusion process are available for corrosion initiation [8].

The time for corrosion deterioration after initiation is currently under debate among researchers. Although the time for corrosion damage for bare bar is generally accepted to be approximately four to six years, less is known about the time for corrosion damage for ECR. Field studies have estimated one to seven additional years for the time to corrosion damage for ECR.

6. Conclusions

The following conclusions are made based on the obtained results:

1. An important aspect of the management and maintenance functions of reinforced concrete bridges which are subject to chloride induced corrosion and deteri-

oration is projection of, first, the present condition and, second, the rate at which continued deterioration can be expected to occur. Only from this information can be optimized decision process be developed for repair, rehabilitation, and, ultimately, partial or full replacement.

2. Existing models that predict the time till first repair and rehabilitation of bridge decks subject to chloride induced reinforcement corrosion can be modified to incorporate the natural variability associated with bridge deck construction, environmental exposure conditions, and reinforcement corrosion using statistical computing techniques.

3. For the concrete bridge the time till first repair and rehabilitation predicted by the probabilistic method was shorter than the time till first repair and rehabilitation predicted by the average value method. The difference was primarily attributed to the variability of the apparent diffusion coefficient and the sensitivity of the time till first repair and rehabilitation to the apparent diffusion coefficient. For input variables with low coefficients of variation, the average value method and the probabilistic methods provide results that are similar.

4. The normalized mean time for diffusion to corrosion initiation of 12% of the steel for the bridges, excluding those with very low surface chloride concentrations, is of 43 years. The mean time to rehabilitation depends on the time for corrosion deterioration for ECR.

5. The utility of the model lies in its ability to estimate the service of a multitude of corrosion protection methods with minimum measurable parameter requirements. Model parameters requirements are effective chloride diffusion constant, chloride corrosion threshold limit, concrete cover depth and the surface chloride content and boundary conditions. The time till corrosion cracking from initiation of corrosion can be estimated as being about five years.

6. The service life model for the corrosion in chloride laden environments was developed for concrete bridge decks and can be used with good results in our country.

Received, April 13, 2005

"Gh.Asachi" Technical University, Jassy,
Department of Foundations,
Roads, Railways and Bridges

REFERENCES

1. Kirkpatrick T.J., *Impact of Specification Changes on Chloride Induced Corrosion Service Life of Virginia Bridge Decks.* Diss. Proposal in Civil and Environm. Engng., Virginia Polytechn. Inst. and State Univ., 2001.
2. Weyers R.E., *Corrosion Service Life Model Concrete Structures. Repair and Rehabilitation of Reinforced Concrete Structures.* The State of the Art Proc. of the Internat. Seminar, Workshop and Exhibition, Maracaibo, Venezuela, April 28 – May 1, 1997.
3. Brown M.C., *Evaluation of the Corrosion Protection Service Life of Debonded Epoxy Coating on Reinforcing Steel.* Diss. Proposal in Civil and Environm. Engng., Virginia Polytechn. Inst. and State Univ., 2000.
4. Bury K., *Statistical Distributions in Engineering.* Cambridge University Press, Cambridge, UK, 1999.
5. Clear K.C., *Effectiveness of Epoxy-Coated Reinforcing Steel, Final Report.* Canadian Strategic Highway Research Program, Ottawa, Ontario, 1998.

6. Covino Jr.B.S., Cramer S.D., Holcomb G.R., Russell J.H., Bullard S.J., Dahlin C., Tinnea J.S., *Performance of Epoxy-Coated Steel Reinforcement in the Deck of the Perley Bridge.* Ministry of Transporation, Ontario, Canada, 102, 2000.
7. Gannon E.J., *A Life Cycle Cost Model for Use in the Optimization of Concrete Bridge Deck Repair and Rehabilitation.* M. Sc. Diss. in Civil Engng., the Pennsylvania State Univ., USA, 1998.
8. Kranc S.C., Sagues A.A., Presuel-Moreno F.J., *Decreased Corrosion Initiation Time of Steel in Concrete Due to Rebar Obstruction of Diffusional Flow.* ACI Mater. J. (submitted).
9. Matsushima M., Tsutsumi T., Seki H., Matsui K., *Study of the Application of Reliability Theory to the Design of Concrete Cover.* Mag. of Concr. Res., **50**, *1*, 5-16 (1998).

MODEL PROBABILISTIC DE PREDICŢIE A PROCESULUI DE COROZIUNE A PODURILOR DIN BETON EXPUSE ACŢIUNII FONDANŢILOR CHIMICI DE DEZGHEŢ

(Rezumat)

Un model probabilistic care să permită predicţia momentului optim de aplicare a primelor lucrări de reparaţii a plăcii podurilor din beton expuse acţiunii fondanţilor chimici aplicaţi pentru dezgheţ pe timpul iernii, trebuie să cuprindă următoarele faze ale procesului de coroziune: procesul de difuziune în profunzime a procesului de coroziune, coroziunea armăturii în intervalul de timp de la efectuarea primelor lucrări de întreţinere până la fisurarea betonului; exfolierea betonului până la nivelul de degradare care să corespundă pierderii capacităţii portante a plăcii podurilor din beton.

Lucrarea prezintă un model probabilistic de predicţie a iniţierii şi dezvoltării unui proces de coroziune la nivelul plăcii podurilor din beton expuse acţiunii fondanţilor chimici de dezgheţ. Aplicarea acestui model are drept scop de a indica cu o precizie suficientă, administratorilor infrastructurilor de transport din România, declanşarea procesului de coroziune şi deci a timpului necesar pentru efectuarea primelor lucrări de întreţinere şi de reparaţii.