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Testing and Modelling to Assess the Capacity of Prestressed Bridges
Expérimentation et modélisation pour évaluer la capacité de ponts précontraints
Modellierung und Versuche zur Einschätzung der Kapazität vorgespannter Brücken

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SUMMARY
Newer assessment methods, aided with theoretical interpretation models, were developed and successfully used on a number of aging and often damaged bridges, with or without transversal cracks. No external test-loads are needed. Experimental results proved consistent and were compared with those calculated according to several models on instantaneous and time-dependent prestress losses. The more recent models better agree with experiment and enable to predict the future remaining structural capacity of a bridge.

RÉSUMÉ
De nouvelles méthodes d'auscultation, aidées par des modèles théoriques d'interprétation, ont été développées et utilisées avec succès pour évaluer la précontrainte résiduelle dans de nombreux ponts en béton, anciens et souvent endommagés, avec ou sans fissures transversales. L'auscultation n'exige aucune charge extérieure. Ces mesures se sont avérées cohérentes. Elles ont été comparées aux résultats obtenus par plusieurs modèles de calcul réglementaire définissant les pertes instantanées et différées de précontrainte. Les modèles les plus récents concordent mieux avec l'expérience, permettant de prévoir l'évolution de la capacité résiduelle future d'un ouvrage.

ZUSAMMENFASSUNG
Neue Untersuchungsmethode mit Hilfe von theoretischen Interpretationsmodellen wurden entwickelt und mit Erfolg eingesetzt, um die restliche Vorspannung bei den älteren und oft geschädigten Betonbrücken mit bzw ohne durchgehende Risse einzuschätzen. Keine Prüfbelastung ist notwendig. Die Messungen haben sich als richtig erwiesen. Sie wurden mit den Ergebnissen verglichen, die man aus mehreren regulären Rechnenmodellen erhalten hatte, und die sofortigen sowie zeitlich verschobenen Vorspannungsverluste definieren. Die neueren Modelle stimmen besser mit Versuchen überein und erlauben so die zukünftige Restkapazität der Brücke vorauszubestimmen.

1. INTRODUCTION

The initial state and evolution of the stress profile is the major parameter of structural reserves in prestressed concrete bridges. A reduced prestressing force may jeopardize their safety. In aging and often damaged structures, the remaining prestress is very difficult to compute with a sufficient accuracy, owing to uncertainties about initial frictional losses, time-dependent hydric and visco-elastic properties of concrete, bond redistribution, random steel corrosion failure and concrete cracks.

With the continuing development of this type of construction, several theoretical models were successively proposed to predict prestress losses. These models need validation or amendment in the light of experiments involving direct stress measurement. In return, modelling may guide the experiment and better interpret its results.

To evaluate the remaining capacity of structures, the present methods of field investigation do require external test loading and indirectly estimate the corresponding stress. If transversal concrete cracks already exist, their adequate intrumentation would provide a better control of test loads. On the other hand, in the absence of cracks and with no prior knowledge of the actual absolute stresses, test loading may become arbitrary and destructive, somehow beating its own purpose. Hence it was necessary to develop new parallel assessment methods that can do without test loading, measure directly the existing stress and apply in both the presence and absence of individual cracks.

2. ASSESSMENT METHODS REQUIRING EXTERNAL TEST LOADS.

For continuous prestressed structures with flexural cracks, experimental computer-aided methods were developed to assess the actual mechanical state and behaviour under dead and live loads.

2.1 Flexural Reserves under Dead Load

For a cracked section, under a convenient gradually increasing test load, figure 1 shows the tendon over-tension variation $\Delta\sigma_a$ with the applied moment ΔM, [1]. This curve, reflecting a reinforced concrete behaviour model, is a reliable criterion of the crack opening. Let X be the point on the curve representing the state of the crack under the dead load of the tested structure ; let I be the crack opening point. The load testing objectif will then be to locate the position of X with respect to I, and determine the bending moment reserves or deficit in the section. If the crack is completely closed under dead load, X will lie on the straight portion around A ; the applied moment experimentally necessary to open it (reaching I) gives the remaining flexural capacity. If the crack is already open, X will lie on the curved portion. In this case, a general way is first to trace, for the same test load, the theoretical strain-moment curve of the tendon $\varepsilon_a = f(M)$, similar to figure 1, using the above-mentioned reinfored concrete model. Where the experimental curve coincides with a portion of the theoretical one, the origin of the former will be the point X and its relative position with respect to I will give the flexural deficit.

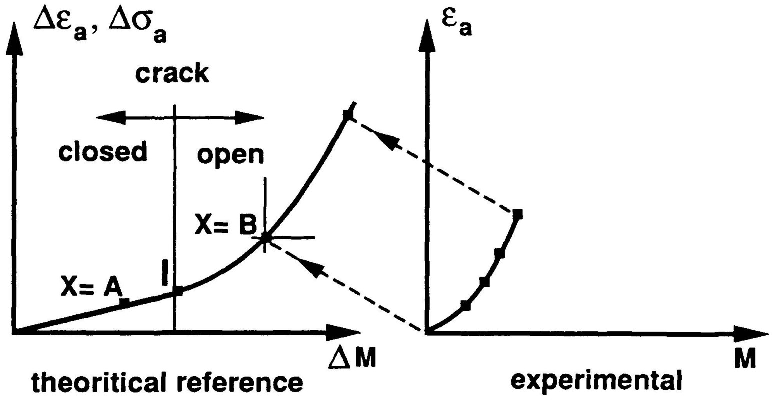

Fig.1
Theoretical and experimental tendon strain and stress variation with the applied bending moment.

2.2 Prediction of Flexural Stresses under Live Loads

In statically indeterminate structures, transversal cracks may be assimilated to a series of elastic or plastic hinges, alternating with sound beam segments and jointhy setting up a new system in equilibrium [2]. The hinge residual flexural stiffness is a multi-parameter function, subject to major

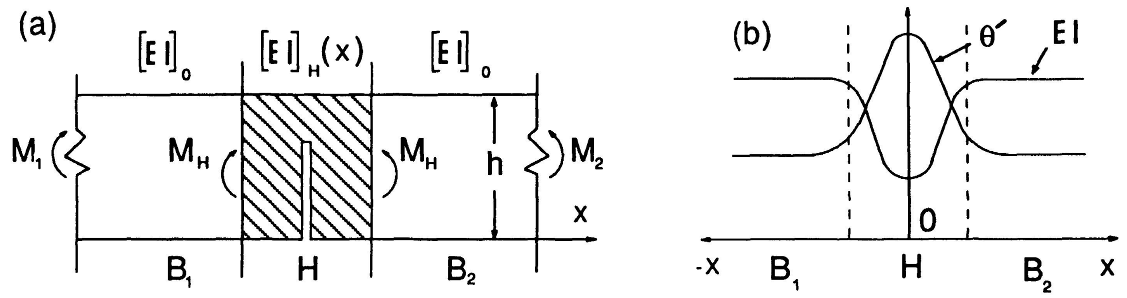

Fig.2 (a) Moments M and stiffnesses EI in cracked sections H and sound segments B
(b) Redistribution of curvature θ' and stiffness EI

assumptions. An experimental evaluation was therefore adopted as follows : Under given test loads, the curvature redistribution is accurately measured throughout the spans. With the flexural stiffnesses of the sound segments usually known, a chain application of the classical beam equation leads to the bending moments and, chiefly, to the required actual stiffness functions of the hinges (Fig 2). These are introduced into a structural analysis program and the new statical system of the bridge is defined. The actual flexural stresses can thus be predicted throughout the structure, under any given live loads.

3. THE CASE HISTORY

In the 1950's, France witnessed a construction wave of prestressed simply supported concrete girder bridges, known by some as the "first generation". Confident of prior tests and careful workmanship, designers pushed the still improving materials to the verge of their performance limits.

Three decades later, hardly any concrete cracks were observed. However, the unfavourable effects of time remained almost unknown on prestress loss, but clearly materialized in severe steel corrosion with local failure of wires.

With a completely uncertaint stress profile and in the still lucky absence of concrete cracks, structural assessment through external load testing connot be recommended for the afore-mentioned reasons.

4. NEW ASSESSMENT METHODS WITHOUT EXTERNAL LOADING

In a fundamentally different approach, we developed two new parallel methods of field investigation. One can directly measure the actual stress in concrete, the other in steel. Both are now successfully applied on site. No test loads are needed.

4.1 Direct Stress Measurement in Concrete ; The Release Method

It is a local and partial release of stress, followed by a controlled pressure compensation [3]. In pratice (figure 3), a displacement reference field is first set up on the concrete surface ; a tiny slot, 4 mm wide, is then cut in a plane normal to the desired stress direction ; finally, a special very thin flat jack is introduced into the slot and used to restore the initial displacement field. The amount of cancelling pressure gives the absolute compressive stress normal to the slot. In the same way, with the same accuracy, tensile stresses are obtained by a corollary. The stress profile is traced by repeating the operation at closely successive depths of the same slot, then by treating the data numerically. The depth operating range is 80 mm. In spite of imposing a minute working scale, counter measures kept the error within 0.3 MPa. Measurement is "direct" in the sense that the same physical quantity is involved (pressure for stress) and that none of the material

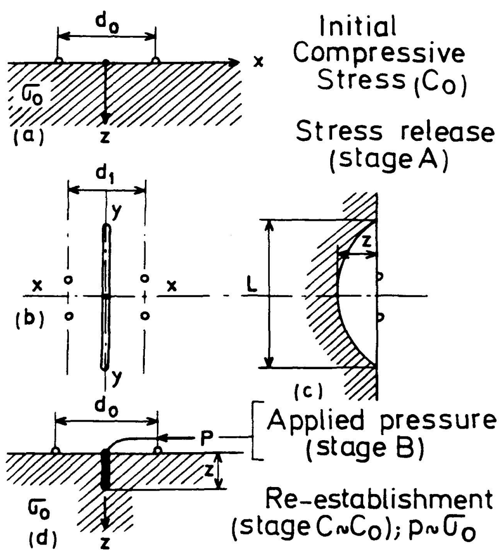

Fig. 3
Stages A, B, C of direct stress measurement in concrete by the release method

elastic properties are needed. These are even determined in the process. Material eigenstresses can now be correctly isolated and subtracted from the measured absolute stress, giving access to the mechanical normal stress acting on the section. Besides the miniaturization imperative, a post-operation specific remedial technique also restores the initial mechanical and esthetic state of the medium. The release method can thus be classified as non-destructive.

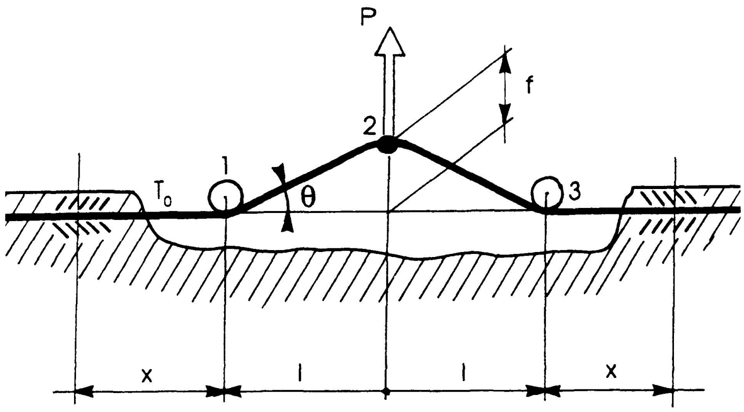

Fig. 4
Pinciple of stress measurement in steel by the crossbow method

4.2 Stress Measurement in the tendons ; The Crossbow Method

The effort necessary to deflect a tight rope is obviously proportional to the axial tensile stress. Figure 4 outlines the application of this fact to the assessment of tendons after carefully clearing the adjacent concrete cover, duct and groute over a 60 cm length [4]. A controlled perpendicular force P, coupled with a displacement sensor, deflects successive prestressing wires throught a distance f limited to 4 mm ; the tensile force F in the steel is deduced by the formula :

$$P = 2\,(F + k)\,(f/l) + K\,(f/l)^3$$

where k and K are given constants. In practice, the parasitie effects of friction, flexural stiffness and overstretching necessitate prior calibration tests on simulating models in the laboratory.

4.3 Other remarks

The release method remains mechanically more comprehensive. Both methods, however, are complementary. They enable now to reason directly in terms of stress thus providing an immediate acces to the applied forces and moments and offering a straightforward comparison with the material strength, a main criterion of structural safety.

5.APPLICATION TO THE CONSIDERED CASE

The two new assessment methods were jointly used to evaluate the remaining prestress in a large number of the simply supported, post-tensioned concrete bridges described in chapter 3 and situated in the north eastern part of France. These structures are 22 to 38 years old, but of identical design and construction : same standard T-sections and spans, same materials and prestressing system.

5.1 Procedure

Concrete stresses at midspan were measured on the neutral axis to deduce directly the actual prestressing force. Steel stresses were evaluated near this section at convient points. Before and during the campaign, thorough inspection was carried out to detect steel corrosion. Affected bridges were left for a separate consideration. On the 12 remaining identical bridges still free from corrosion, the stress values obtained by both methods were quite consistent and fully agreed. They were selected for the following analysis on the mechanical loss of prestress.

5.2 Experimental Data Synthesis

In a homogeneous group of N bridges, solely differing in age, the N respective stresses measured at a given date may be assumed equivalent to the measurements that could have been taken on a single typical bridge at N consecutive dates of its lifetime. In our case, the 12 corresponding stress values can thus form a synthesis curve representing an experimental time-dependent loss of prestress.

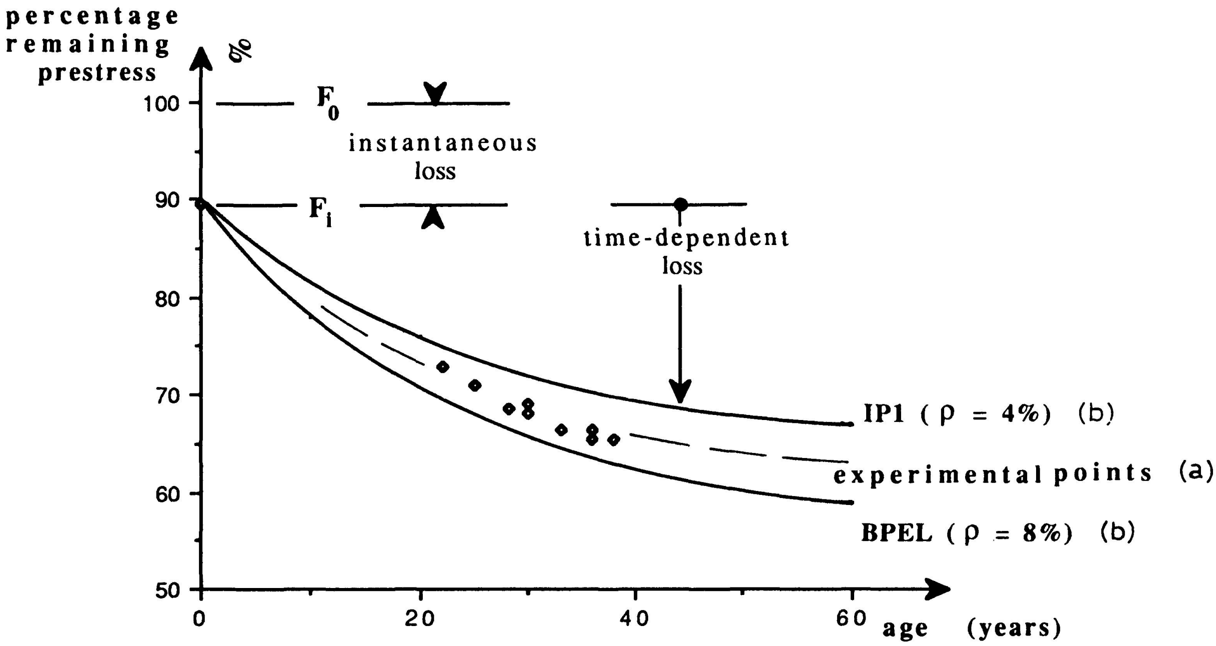

Fig. 5

a) Directly measured remaining prestress at mid-span, on 12 identical bridges of different age and with unknown steel relaxation ρ and friction coefficient μ.

b) Corresponding envelop theoretical curves obtained from two predictive models with different basic assumptions and ρ values ; they help to extrapolate the experimental results in time and determine ρ for the steel actually used ; experiment may in return improve the model assumptions.

In figure 5, these points are converted to forces over the section and expressed as a percentage of the original anchorage prestress force F_0 found in the records of each bridge. They lie between two theoretical envelop curves, stemming from constitutive models discussed below, and guiding the extrapolation of the present measurements towards the future (or the past).

6. THEORETICAL MODELS ON PRESTRESS LOSS

Three main frameworks of prestress loss estimation were proposed at sucessive periods of construction development in France : pre-code recommendations, IP1 model, BPEL code.

6.1 Instantaneous losses

The effects of friction, elastic shortening and anchorage loss are all well known. The main uncertainty concerns the values of the friction and wobble coefficients, μ and ϕ, in the exponential formula which all models use for frictional losses.

6.2 Time-dependent losses

On prestress losses due to concrete shrinkage and creep or steel relaxation, the three models diverge as follows.

* The pre-code of the1950's, construction period of the bridges now under scrutiny, gives a lump sum estimation of 12 to 15 % prestress loss in all, after the instantaneous effects [5]. It assumes proper overtensioning to reduce steel relaxation and overcome friction and anchorage losses.

* In the IP1 model of the 1960's, based on the concept of allowable stresses, a group of constitutive equations accounts for the following sources of loss [6] :

- Concrete shrinkage, depending on the mix and thickness, the atmospheric humidity and a time-dependent strain coefficient.

- Concrete creep, as a function of the initial elastic strain, atmospheric humidity, time-dependent strain coefficient and other constitutive factors.

- Steel relaxation, depending on the initial steel stress σ_i, strength R_g and percentage stress loss ρ at 1000 and 3000 hours in a standard relaxation test.

The main assumption in IP1 is the simple straightforward superposition of these three.

* The B.P.E.L., a more elaborate model of the 1980's, is based on the service and ultimate state limits [7]. Compared with the IP1, there is hardly any difference as far as concrete shrinkage. For concrete creep, in a new BPEL version : The effect of atmospheric humidity is more accurately formulated in the basic law ; an additional but different law was introduced for "creep recovery" ; a new "equivalent time" method was developed to replace the superposition principle.

Concerning steel relaxation, the same IP1 parameters were kept in a slightly different formula, less optimistic for medium relaxation steel used in the bridges under consideration.

BPEL's basic assumption is the partial interaction between concrete creep and steel relaxation.

7. MODEL-AIDED INTERPRETATION OF EXPERIMENTAL RESULTS

For both the IP1 and BPEL predictive models, detailed computer programs were elaborated [8]. Given the material properties, they calculate all the prestress loss compoments in an exact step-by-step simulation of the bridge construction phases and loading history. For the assessed bridges, the properties related to the calculation of concrete shrinkage and creep were either given or deduced from the design and construction records. However, the percentage steel relaxation at 1000 hours, ρ, did not exist as such in the early 1950's ; it was actually very high but masked by short-time tests. Another uncertainty was μ, the prestress friction coefficient. In a parametric study using both models, ρ was varied from 4 to 8% (extreme value), and μ from 0.16 to 0.23. The analysis showed that ρ had an effect several times greater than μ on the total prestress loss. Hence, we thought of fixing μ at 0.23, corresponding statistically to the type of ducts used. Left with one influential unknown material parameter ρ, it was not possible to have a clear-cut comparison between experimental and theoretical results but rather a joint contribution : the release method giving the present actual prestress force, the two models guiding its extrapolation to the future under a ρ variation range suggested by previous data synthesis and living memory. In this context, the adopted interpretation strategy is illustrated in figure 5 for the 12 assessed bridges, then explained in table 1 by an example of a typical 30-year-old bridge. In the example, interpretation proceeds as follows :

1) From the anchorage prestress force F_0 given in construction records, calculate by each model the remaining mid-span force F at infinity (∞ = 60 years), using two envelop values of ρ and the adopted constant value of μ. (M1, M2 and M4, M5 in table 1).
2) Deduce the present force F (30 years), assuming a continuons quasi-asymptotic function of time.
3) Compare the present experimental value of force F with the two theoretical ones of each model.
4) By identification or interpolation, find in each model a particular theoretical function that agrees with the present measurement (5500 KN at 30 years). The functions obtained nearly coincide and can predict the prestress losses still expected.

Thus, at 30 years, the prestress of the bridge, reflecting its structural capacity, has already actually suffered a 32 % total loss, expected to attain 38 %. Of these, 27 % are time-dependent contrasting with the 15 % estimated in the design. The two particular functions, satisfying the 30-year stress measurement and giving the same losses, correspond to ρ = 8% for IP1 and only 6.27 % for BPEL. Despite the lessening effect of its partial interaction between concrete creep and steel relaxation, BPEL seems then to grow passimistic for ρ values exceeding the 2 % of our present low relaxation steel. Finally we notice practically stable values of instantaneous losses, 10-11 %, appearing in the corresponding differences between the last two columns of table 1.

8. UP-TO-DATE ACTIONS ON THE STRUCTURE

With its remaining capacity evaluated, the structure must face external loads not necessarily équivalent to those its was designed for. The Eurocode will thus be proposing two partial load systems :

- A double-axle concentrated load model (indivisible tandem system), each axle having a weight Q_k

Means of Evaluation: modelling / Experiment	Prestress Force F (KN)		Steel Stress σ_a (MPa)		Concrete Stress σ_b (MPa)		Time-dept % Loss $1 - F/F_i$		Total % loss $1 - F/F_o$	
	30 y	∞	30 y	∞	30 y	∞	30 y	∞	30 y	∞
Pre-code : M 0 approximation (design)	6430	5900	994	912	9.0	8.3	8	15	20	27
IP1 model : M 1 $\rho = 8\ \%$; $\mu = 0.23$	5500	5040	850	779	7.7	7.1	21	28	32	38
IP1 model : M 2 $\rho = 4\ \%$; $\mu = 0.23$	5800	5400	897	835	8.1	7.6	17	22	28	33
IP1 model : M 3 $\rho = 8\ \%$; $\mu = 0.16$	5600	5090	866	787	7.8	7.1	19	27	31	37
BPEL code : M 4 $\rho = 8\ \%$; $\mu = 0.23$	5340	4760	826	736	7.5	6.7	23	32	34	41
BPEL code : M 5 $\rho = 4\ \%$; $\mu = 0.23$	5710	5260	884	814	8.0	7.4	19	24	29	35
EXPERIMENT (Direct Stress measurement)	5500	—	850	—	7.7	—	21	—	32	—
Identification with M 1	5500	<u>5040</u>	850	<u>779</u>	7.7	<u>7.1</u>	21	<u>28</u>	32	<u>38</u>
Interpolation M 4 - M 5 $\rho \rightarrow 6.27\ \%$	5500	<u>4980</u>	850	<u>769</u>	7.7	<u>7.0</u>	21	<u>28</u>	32	<u>38</u>
Conclusion :	The bridge may still lose 10 % of its present prestress.									

<u>Table 1</u> : Example of an experimental model-aided assessment of present and future prestress losses in a bridge, 30 years old at the testing date. Underlined quantities are experimental values extrapolated to infinity using these models (ρ is the steel relaxation at 1000 hours, μ the friction coefficient , ∞ assumed at 60 years. F is the present and future prestress force at mid-span ; F_i and F_o are the initial forces respectively at mid-span and the anchorage).

equal to 300, 200, 100, KN on lane 1, 2, 3, respectively.
- A uniformly distributed load (UKL system), having a weight density q_k per square metre, applied in the unfavourable areas ; q_k is assumed 9, 2.5, 2.5, 2.5 KN/m^2 on lane 1, 2, 3, elsewhere.

10. RANDOM EFFECTS OF CORROSION

On the second group of bridges, the experimental results are far less consistent. The impact of steel corrosion failure, on the remaining prestress in nearby sections, varies from negligible to considerable, depending on the ability of the broken wires to develop local friction anchorages in both senses (fig. 6). This random factor is still difficult to model. Direct stress measurement, by the release method in particular, remains the only unchallenged means of assessment for the time being.

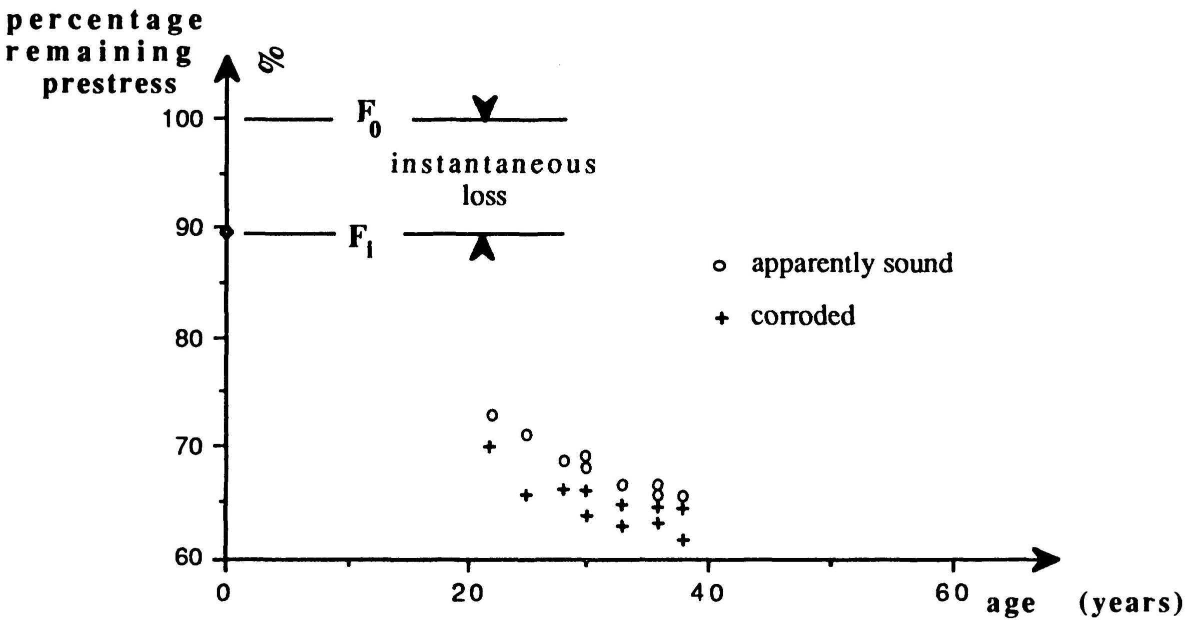

Fig. 6
Effect of steel corrosion on the remaining prestress of otherwise identical bridges.
Both groups seem to have undergone similar time -dependent losses.

10. CONCLUSION

The present actual structural reserves of a prestressed concrete bridge can now be evaluated through direct stress measurements by the release method. A model-aided extrapolation of these stresses gives the time-dependent losses still expected and, hence, the future residual capacity. Proposed load models, such as "Eurocode 1", will be contributing to an up-to-date estimation of the total required capacity of a structure. The difference between the required and remaining reserves leads obviously to the needed additional external prestress. At a later stage, stress can again be measured to check the effectiveness of the operation. The release method on one hand, two predictive models on the other, can thus jointly contribute to an optimum strengthening of aging bridges, which inadequate prestressing has rendered structurally deficient or obsolete.

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