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Unexpected Fatigue Failures of Non-prestressed Reinforcements

Rupture inattendue de l'armature passive due à la fatigue

Unerwartete Ermüdungsbrüche in der schlaffen Bewehrung

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SUMMARY
The calculation models to predict the response of partially prestressed concrete bridge girders, subjected to traffic cyclic loads, usually incorporate many assumptions about the pattern behavior. One of the most widely used is the perfect bond between steel and concrete, neglecting the local bond breakdown surrounding each flexure crack. Laboratory test results given in this paper show that some unexpected fatigue failure, due to cumulative damage in non-prestressed reinforcement, can be found. Fretting secondary stresses, owing to the curvature at deflection, ought to be included in a Class 2 and Class 3 design.

RESUME
Les modèles de calcul pour estimer la réponse d'une poutre d'un pont en béton partiellement précontraint, soumise à des sollicitations dynamiques dues au trafic, contiennent généralement beaucoup de simplifications sur le comportement réel des éléments structuraux de ces ouvrages d'art. Une des hypothèses les plus utilisées est la parfaite adhérence acier-béton, malgré son inexistence à proximité de la fissure. Cet article donne les résultats d'essais montrant quelques ruptures dues à des dommages cumulatifs de l'armature passive. Il faut tenir compte dans le calcul mathématique en Classe 2 et 3, des contraintes secondaires de friction dues à la courbure de l'armature passive.

ZUSAMMENFASSUNG
Die Berechnungsmodelle über das Verhalten teilweise vorgespannter Betonbrückenträger unter zyklischer Verkehrsbelastung enthalten oft viele Vereinfachungen gegenüber dem wirklichen Verhalten. Eine der häufigsten Vereinfachungen betrifft die Vernachlässigung der unvollständigen Haftung zwischen Stahl und Beton in Rissnähe. Die im Beitrag vorgestellten Versuchsergebnisse zeigen unerwartete Ermüdungsbrüche in der schlaffen Bewehrung. Die durch die Biegekrümmung hervorgerufenen Nebenspannungen infolge Reibungseffekten sollten in den Berechnungen für die Brückenklassen 2 und 3 berücksichtigt werden.

1. INTRODUCTION

A research investigation, conducted in the Department of Structures at the Laboratorio Central de Estructuras y Materiales, is being carried out to obtain information on the fatigue life of prestressed concrete bridge beams under traffic cyclic loads. Full size bridge girders have been tested. However, in order to attain an acceptable level of reliability, further beam tests are being made to explain some specific questions. A knotty point was the verification of fatigue behavior of the non-prestressed steel addition (commonly advised against fatigue failure) after cracking. As a matter of fact that situation appears not only in a Class 3 design, but in Class 2 too, for exceptional live loads. This feature was included in the test program of series E2, which the present article deals with. On the other hand, the choice of a good mathematical simulation model, to find out the stresses in those reinforcements, is not an easy task, especially if we incorporate some usual assumptions. With regard to that question, as the present work shows, we might be on the unsafe side.

2. SIMULATION MODEL AT CRACKED SECTION

2.1 Hypothesis for simplification

Technical literature contains many theoretical and experimental works on the behavior of beams bending under repeated overloads. When the external moment exceeds the value at which cracks begin to open and close, their flexural response is non-linear and conditions become more complicated. Nevertheless the computational effort can be reduced if the following assumptions are made:

- Strains vary linearly over the depth of the member throughout the entire load range.
- After cracking, tension in the concrete is neglected.
- Equilibrium of internal forces.
- Linear elastic response of steel instead a more complicated (history dependent) constitutive relation. During the first cycles of loading, this behavior is not true if that material has been stressed up to yield, and some analytical model for steel should be used.
- The perfect bond between steel and concrete seems to be the most doubtful hypothesis in this model. Some authors bear in mind the bond failure introducing a dimensionless compatibility factor. However, the factor evaluation is difficult because of the scatter natures of this phenomenon and values must therefore be determined from beam tests.
- Effects of shear are negligible on sufficiently slender members.

2.2 Striped model

As can be seen from Figure 1, the model is valid for any section shape, with steel (either prestressed or not) reinforcements at several different levels. To compute the integrals appearing in the mathematical equations, the compressive block of section is divided into vertical strips. In every one of them a cubic parabola was approximated from the concrete stress-strain curve (Figure 2) in order to provide the compressive stress distribution. The flow chart in Figure 3 illustrates, step by step, the program for computer which allows the stresses evaluation. [2] [3]

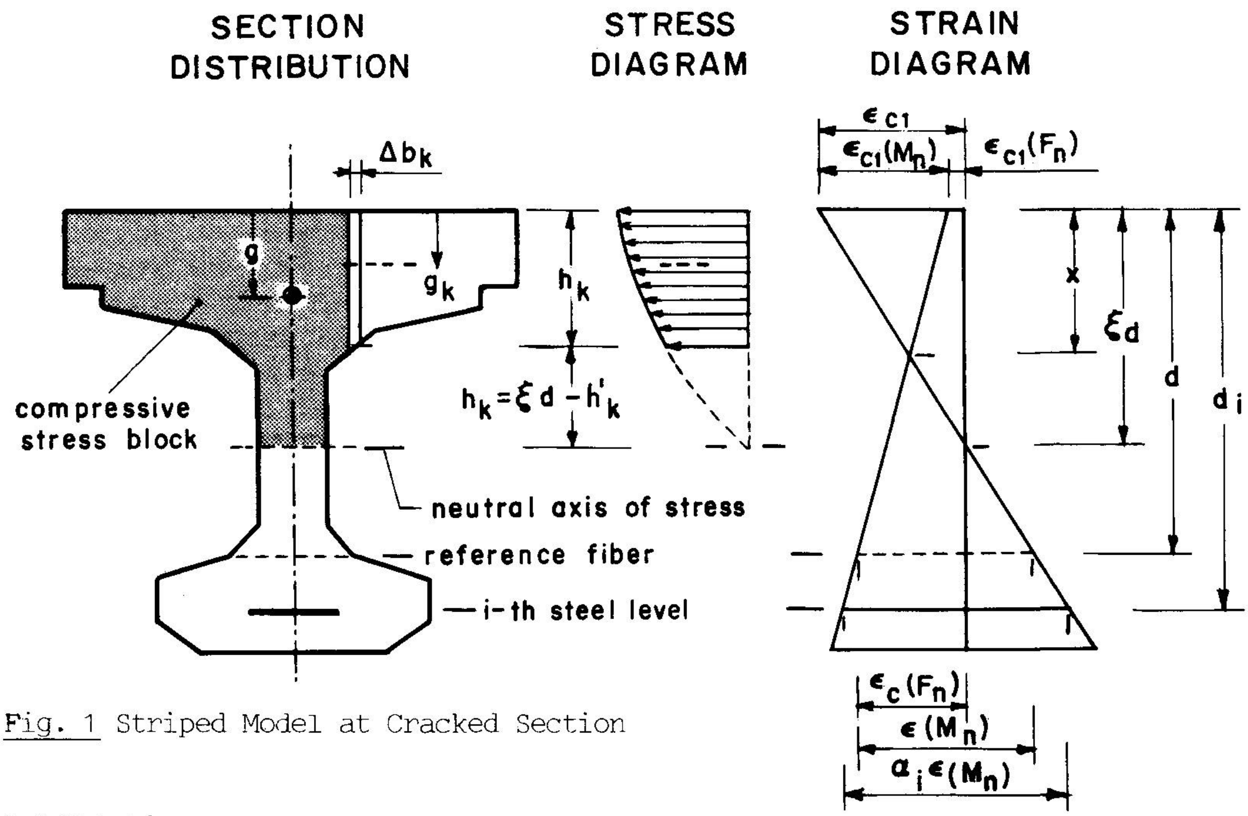

Fig. 1 Striped Model at Cracked Section

2.3 Notations

The following terms have been used in the present model:

- N - number of steel levels (including non-tensioned reinforcements)
- E_s - modulus of elasticity of steel
- A_i - area of longitudinal steel at i-level (i = 1 to N)
- d_i - depth of i-level (i = 1 to N)
- $f_{si}(F_n)$ - steel stress in the n-th load cycle due to the prestress force at i-level
- F_n - prestressing force in beam during the n-th load cycle
- d - depth of reference fiber
- f_c - concrete cylinder strength
- $\mathcal{E}_u$ - concrete strain in cylinder at f_c
- λ - dimensionless parameter defining the shape of the concrete stress-strain relation
- η - dimensionless quantity relating concrete strength in beam and cylinder
- $\mathcal{E}_{c1}(F_n)$ - concrete strain in top fiber of the beam due to the prestress force
- $\mathcal{E}_c(F_n)$ - concrete strain at reference fiber due to the prestress force
- M_n - applied moment in the n-th load cycle
- $\mathcal{E}(M_n)$ - virtual strain at reference fiber due to M_n

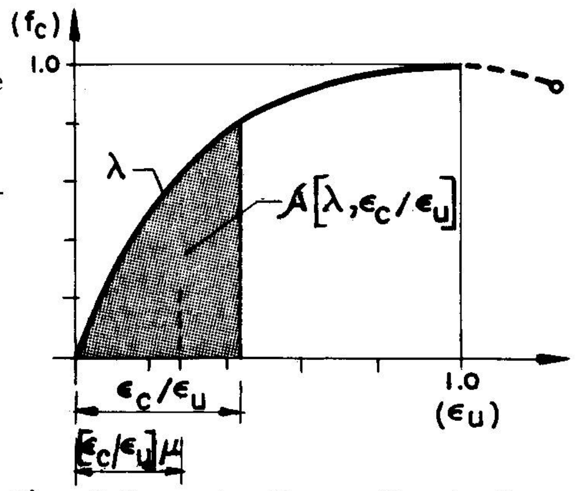

Fig. 2 Concrete Stress-Strain Curve

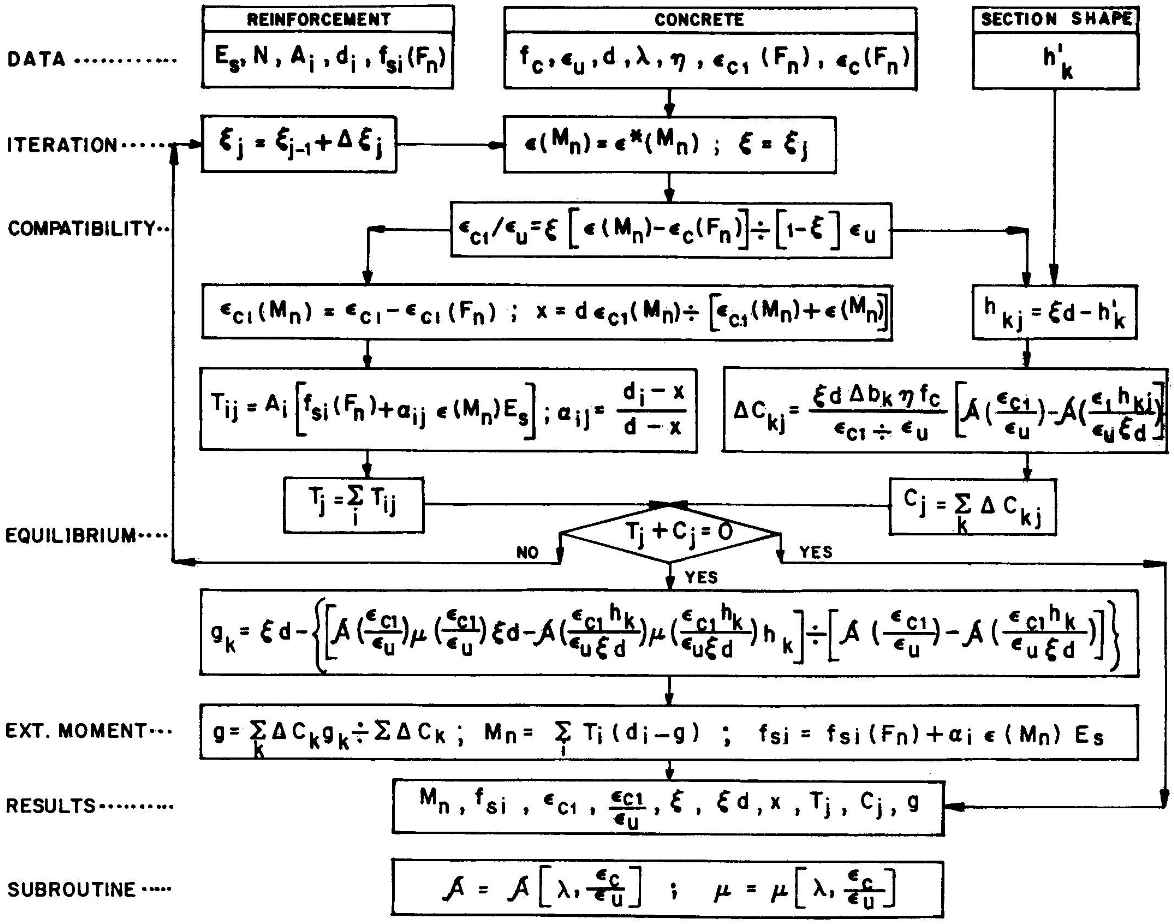

Fig. 3 Flow Chart for Stress-Moment Curve Evaluation

$\varepsilon_{c1}(M_n)$ - concrete strain in top fiber of the beam due to Mn
x - depth of neutral axis of bending
ξ - dimensionless factor defining location of neutral fiber of stress
ξd - depth to neutral fiber of stress
α_i - strain distribution factor
f_{si} - steel stress at i-level
C - total compressive force in concrete

3. SERIES E2 TESTS

3.1 Description of specimens

Each of the 6 m. length T-beams was simply supported. The cross section and the situation of the post-tensioned bonded tendons (level 2) and the non-prestressed reinforcements (level 1 and 3) are shown in Figure 4. The area of concrete section was o.1252 sq.m. and the moment of inertia of this area about its centroidal axis 243 x 10^{-5} m^4. Reference fiber was chosen at center of gravity of level 2 plus level 3. The concrete used in the manufacture of specimens was vibrated and twelve standard 150 mm. by 300 mm. test cylinders were cast with each beam.

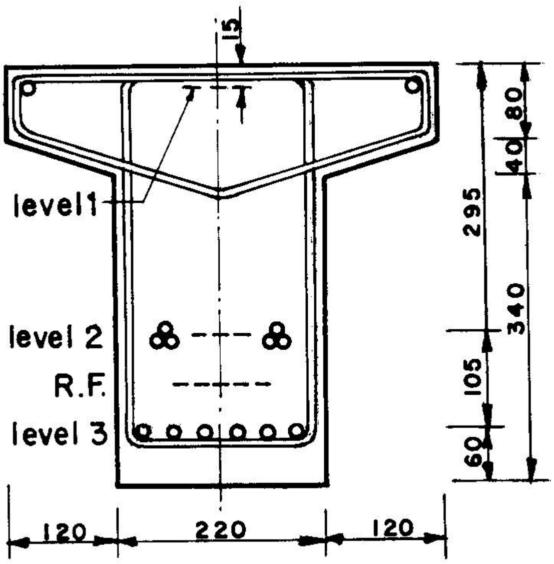

Fig. 4 Beam E2-2 Section

Concrete cylinder strength was about 39 MPa and tensile strength about 2.5 MPa. The stress-strain data for all the beams, without prior loading, follow quite well the λ = 2.6 curve (below 85% of concrete cylinder strength). This value provides a reasonable fit for the compressive stress block of the beams at all load levels. Only plain cold-drawn wire has been considered in series E2 and the ultimate tensile strength for the 5 mm. diameter wire was 1898 MPa. The amount of web reinforcement, given in Figure 4, was just sufficient to develop the ultimate flexural capacity of the beams. In every one of them, prestressing began 30 days after casting. Previously to grout the tendon ducts at level 2, losses of stress due to wedge anchorage, elastic shortening, concrete creeping and steel relaxation, were verified during 20 days after prestressing.

3.2 Test procedure

Every one of the beams was tested up to three million cycles of loading. After which, if the beam went beyond this limit, a static test up to failure would be performed in order to compare the ultimate static strength between beams with different load histories. The main difficulty, during the first (about 10^5) cycles of repeated loading, is that prestress force varies due to creep effects and progressive bond failure. Changes in the compressive stress block are less important as a result of low stresses in under-reinforced members. Losses of - effective prestress provide increasing stresses at non-tensioned wires. Henceforward, the beam settles down to a fairly steady response. Beams were instrumented with exterior strain gauges at steel levels. The measured values of

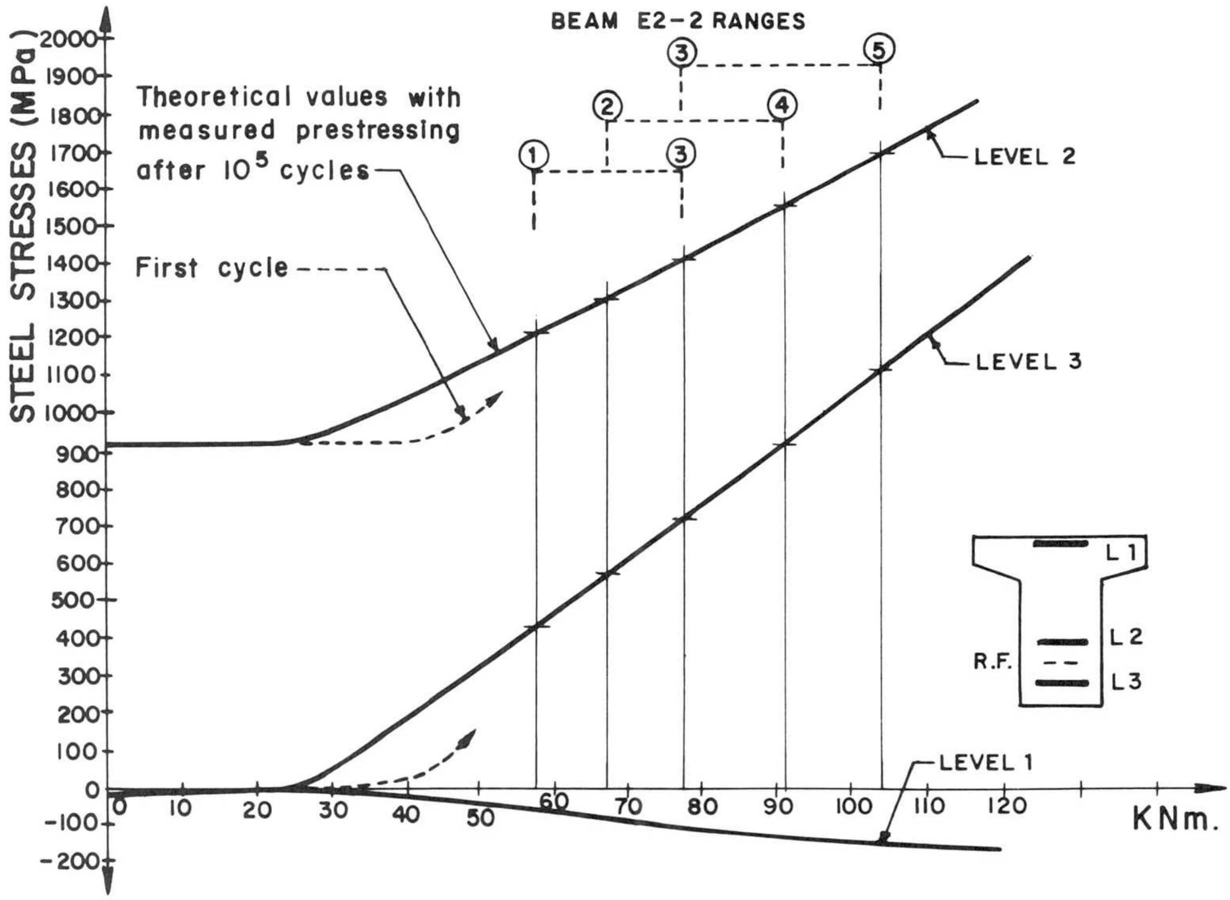

Fig. 5 Reinforcement Stresses vs. Applied Bending Moment

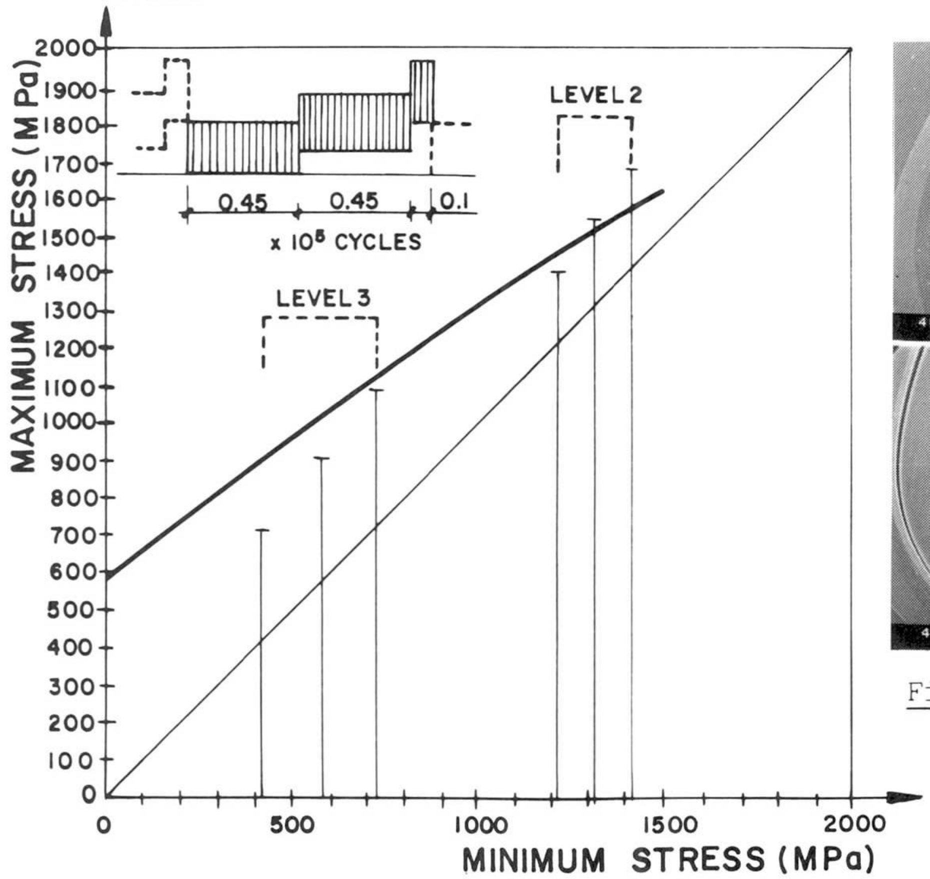

Fig. 6 Goodman Diagram for Steel Reinforcements (Beam E2-2)

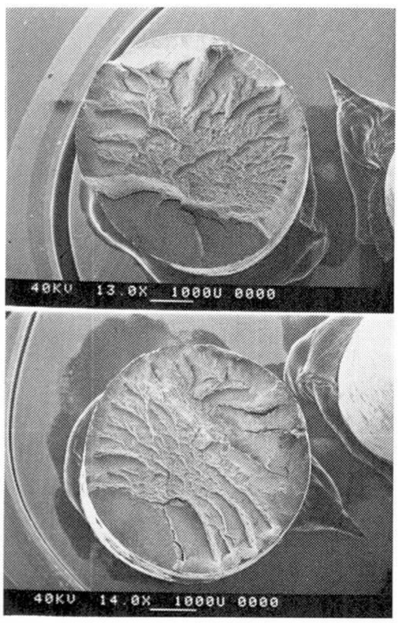

Fig. 7 Wire Fracture Surfaces at Level 3 (Beam 2)

reinforcement stresses agreed very well with the calculated ones plotted in Figure 5, which shows the relationship between those stresses and the applied bending moment. The load spectrum has been divided into three steps and varying from each beam to others. Some stress ranges in prestressed reinforcements crossed the Goodman diagram boundary, in all beams, indicating a risk of failure. However, ranges in non-prestressed wires were kept well within that limit. Figure 6 indicates the loading sequences 1-3, 2-4 and 3-5 of the E2-2 beam cummulative test, which is summarised in the following table:(Theoretical values)

Load	Bending Moment (KNm)	Stress at Level 2 (MPa)	% Ultimate Strength of wire	Stress at Level 3 (MPa)	%Ultimate Strength of wire
1	57.52	1215	64	418	22
2	66.78	1310	69	569	30
3	77.38	1405	74	721	38
4	90.61	1537	81	911	48
5	103.90	1670	88	1101	58

3.3 Test results and discussion

Failure was brought about only in E2-2 beam. As Figure 7 illustrates, typical fracture surfaces, due to cummulative damage, appeared in non-prestressed wires at the bottom layer. Failure took place after 1995 x 10^3 cycles, in spite of theoretical steel stresses, which seem to keep off the modified Goodman diagram boundary. On the other hand, a good agreement between those corresponding strains and the measured ones, proved a fitness of the used model. Therefore it is clear that some kind of supplementary stresses in non-prestressed steel has been neglected. The use of linear elastic fracture mechanics provides an approaching way to estimate the actual maximum stress at failure. Fracture toughness 66.4 MPa$\sqrt{m}$ was taken from several plain cold-drawn wire tests. The equalling of this value and the stress intensity factor average, pointed out a real stress up to 1380 MPa, which really cross through the Goodman diagram limit (26% more than expected). These additional stresses might occur due to an oscillatory rubbing action between mating surfaces near each crack. Although the nature of fretting-induced fatigue is not clearly understood, it is well known that fretting stresses can appear with extremely small relative amplitude (10^{-5}mm). Cracks in the present tests opened up to 0.05 mm wide. [5]

4. FINAL REMARKS

To make use of non-prestressed steel is commonly advised in order to safeguard prestressed beams-under traffic loads-against fatigue failure. It seems to be appropriate because the resistance to cracking is considerably improved. However, when cracks have occured, fretting secondary stresses as consequence of the steel curvature at deflection might take place near each flexure crack. It is early, until series E2 becomes closed, to conclude that some unexpected failure due to non-prestressed steel fatigue could occur if those stresses were overlooked.

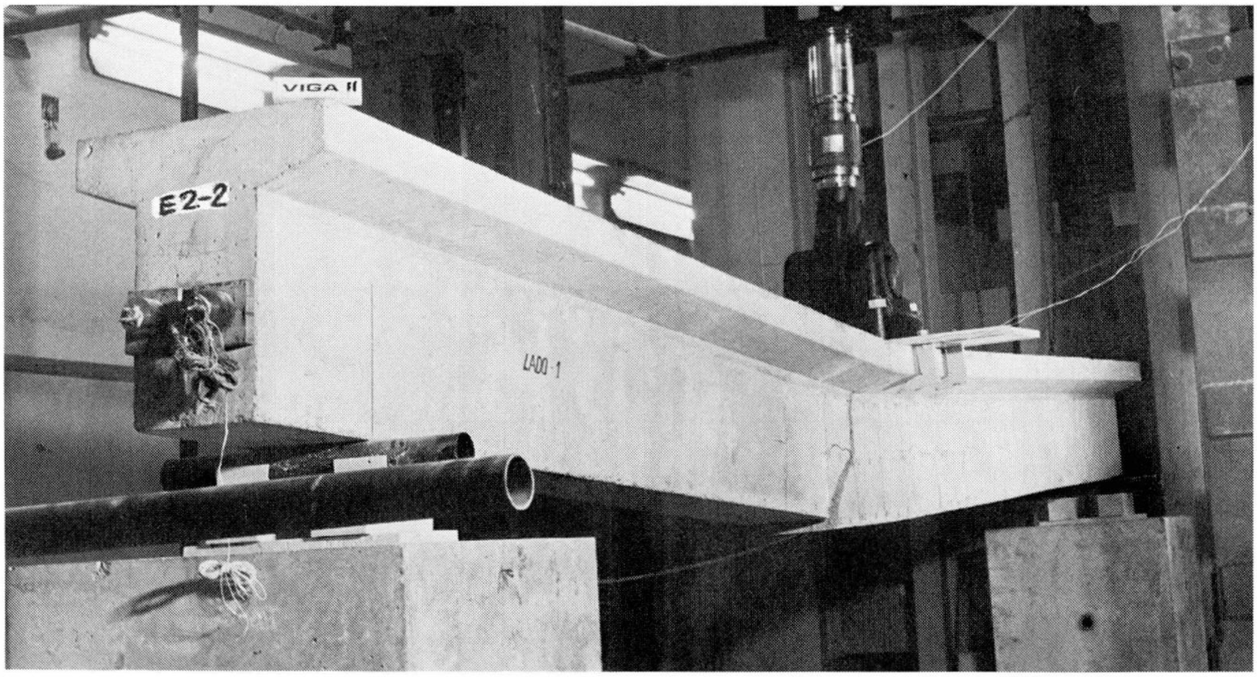

Fig. 8 Photograph of E2-2 Beam After Failure

ACKNOWLEDGEMENTS

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