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Dynamic Tests for Diagnosis of Damage Caused by an Earthquake

Essais dynamiques pour la détermination de dommages causés par un séisme

Dynamische Versuche für die Bestimmung von Schäden durch Erdbebenbeanspruchung

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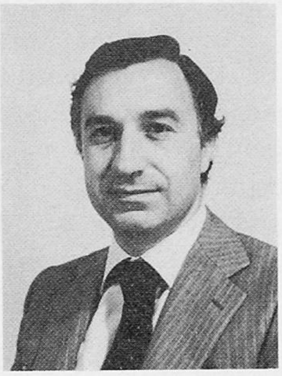

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SUMMARY

The results of dynamic tests carried out on two reinforced concrete piers of a bridge under construction in the area struck by the Irpinia earthquake of November 23, 1980, are illustrated. The experimental determination of the dynamic behaviour of the piers, carried out by excitation through mechanical vibrators, enabled the elastic and dissipative characteristics of the soil to be examined, as well as the soil-foundation interaction effects, and the type and degree of damage of the piers to be evaluated.

RESUME

Les résultats des essais dynamiques conduits sur deux piles en béton armé d'un pont en construction dans la zone frappée par le tremblement de terre d'Irpinia, le 23 Novembre 1980, sont présentés. La détermination expérimentale du comportement dynamique des piles, effectuée par excitation avec des vibrateurs mécaniques, a permis d'examiner soit les caractéristiques élastiques et de dissipation du sol, soit les effets d'interaction sol-fondations et d'évaluer le type et le degré de dommage des piles.

ZUSAMMENFASSUNG

Es werden die Ergebnisse aus dynamischen Versuchen an zwei sich im Bau befindlichen bewehrten Brückenpfeilern aus der Gegend, die am 23. November 1980 vom Irpinia- Erdbeben heimgesucht wurden, vorgestellt. Die experimentelle Untersuchung des dynamischen Verhaltens der Pfeiler, deren Erregung mittels mechanischer Vibratoren erfolgte, erlaubte auch die Ermittlung der elastischen und dissipativen Charakteristiken des Bodens. Ferner konnten auch Boden-Fundament-Interaktionseffekte sowie Typ und Grad des Schadens an den Pfeilern bestimmt werden.

1. INTRODUCTION

After the Irpinia earthquake of November 23, 1980, an inspection to the 12 piers of a highway bridge under construction on the River Calore, showed the presence of some fine horizontal cracks all around the piers, at a height ranging from 3.5 to 4 meters approx. over the foundation block. Following the discovery of these cracks, the problem arose of assessing their importance for the structural safety of the piers, and consequently of estimating their extent and penetration into the cross-section.

For the diagnosis of the damage and for the determination of the overall structural behaviour, an experimental research has been carried out with the aim of identifying the characteristics of the system through the evaluation of the foundation soil parameters and the response of the piers to the external loads. The procedure chosen has been the performing of dynamic tests, by applying at the top of two piers of different height (pier A: 17 m; pier B: 36 m) and foundation conditions (pier A: on piles; pier B: directly on soil) variable frequency sinusoidal forces. The general scheme of the pier is shown in Fig. 1; owing to the different dynamic characteristics of the piers under test two different mechanical vibration generators were used.

The response of the structures has been measured by means of 10 accelerometers (on the shaft of the piers), 6 seismometers (on the foundation block) and 8 strain and displacement meters (across the cracks): by means of these instruments the overall behaviour as well as the local one in points of special interest has been read out.

As output of the tests the time histories of the structure responses at the various different points have been obtained, as well as the transfer functions between these responses and the exciting force.

2. CRITERIA FOR INTERPRETATION OF TEST RESULTS

In order to obtain direct information on the behaviour of the various parts of the structural system, a reference scheme has been set up as shown in Fig. 2, in which the elements taken into account are: the top platform, the shaft, the foundation block and the soil. The response of the system depends on the following quantities:

- masses m_p and m_z, and inertia moments I_p and I_z of the top platform and the foundation block (rigid bodies)
- elastic (A), dissipative (B) and inertial (G) characteristics of the shaft, considered as a flexible element
- elastic (k_u, k_φ) and dissipative (b_u, b_φ) parameters of soil, with respect to displacement u and rocking φ of the foundation block.

The equations of motions of the structures under test can be expressed in terms of absolute displacements, relevant to the system as a whole; in this case the structural response is given - in the frequency domain - by the relationship:

$$Y_a(z, f) = H_c(z, f) \cdot C(f) \tag{1}$$

where $H_c(z, f)$ is the transfer function of the system, that is the ratio of the absolute response $Y_a(z, f)$ and the applied load $C(f)$. $H_c(z, f)$, in turn, depends on the modal characteristics of the system, its expression being:

$$H_c(z, f) = \sum_1^{\infty}{}_j \frac{r_c^{(j)} \cdot \psi_j(z)}{\left[1-\left(\frac{f}{f_j}\right)^2 + 2i\nu_j\frac{f}{f_j}\right]} \tag{2}$$

where: $f_j, \nu_j, \psi_j(z)$ and $r_c^{(j)}$ are respectively the frequency, damping coefficient, modal shape and participation factor of the j-th vibration mode of the pier.

A second way for the analysis of the system, more significant for the aims of the present study, is that of describing the motion (again in the frequency domain) by the following set of equations:

$$Y_r(z, f) = \bar{H}_c(z, f) \cdot C(f) - \bar{H}_u(z, f) \cdot \ddot{U}_o(f) - \bar{H}(z, f) \cdot \ddot{\phi}_o(f) \tag{3}$$

$$\begin{aligned} F_s(f) &= k_u \cdot U_s(f) + b_u \cdot \dot{U}_s(f) \\ M_s(f) &= k_\varphi \cdot \phi_s(f) + b_\varphi \cdot \dot{\phi}_s(f) \end{aligned} \tag{4}$$

In equation (3) the transfer functions $\bar{H}$ contain the modal parameters of the shaft with the top platform, however considered as "clamped" at its base. Equations (4) are relevant to the soil, and express the relationships between the external actions (horizontal forces F_s and overturning moments M_s) and the horizontal motion (U_s and $\dot{U}_s$) and rocking (ϕ_s and $\dot{\phi}_s$) of the foundation.

3. ANALYSIS OF THE RESULTS OF THE TESTS

3.1. Overall behaviour

The response of the two piers tested, as shown by the measures obtained from the accelerometers and the strain meters, is markedly non-linear in both directions of excitation. As an example, Fig. 3 a illustrates the response curves for pier A, whereas Fig. 3 b shows the time history of the top acceleration recorded on pier B, for tests with increasing and decreasing frequency. Both patterns are typical of a "softening" system.

In order to find out the causes of such non-linearity, the further processing of the data has been carried out with the aim of separating the behaviour of the pier from that of the foundation soil.

3.2. Elastic and dissipative characteristics of soil

As to the soil behaviour, its elastic and dissipative characteristics have been obtained from equations (4). Their expressions are the following:

$$k_u = RP\left[F_s(f)/U_s(f)\right] \qquad k_\varphi = RP\left[M_s(f)/\phi_s(f)\right]$$
$$b_u = RP\left[F_s(f)/\dot{U}_s(f)\right] \qquad b_\varphi = RP\left[M_s(f)/\dot{\phi}_s(f)\right] \tag{5}$$

that is the real parts (RP) of the ratios within the brackets.

The quantities U_s, ϕ_s, $\dot{U}_s$ and $\dot{\phi}_s$ are known since they have been measured experimentally by the seismometers placed on the foundation block; in turn, the values of F_s and M_s have been processed on the basis of the response accelerations, the test characteristics and the inertia and geometry of the piers.

The values of k and b obtained from the experimental results are plotted in Fig. 4: they show that the soil behaviour - at least within the limits, rather broad however, reached by the external loads during the tests - is practically linear. It follows that soil cannot be responsible for the highly non-linear behaviour of the systems.

3.3. Soil-structure interaction effects

The evaluation of the soil-structure interaction effects has been obtained directly from the experimental data, by a comparison between the response curve $\overline{H}_c(z, f)$ and the response $H_c(z, f)$ actually recorded. $H_c(z, f)$ is obtained straightforwardly from equation (1):

$$H_c(z, f) = Y_a(z, f)/C(f) \tag{6}$$

whereas $\overline{H}_c(z, f)$ is worked out from equation (3):

$$\overline{H}_c(z, f) = \frac{Y_r(z,f)}{\left[C_f - \frac{\overline{H}_u}{\overline{H}_c}\cdot\ddot{U}_o - \frac{\overline{H}_\varphi}{\overline{H}_c}\cdot\ddot{\phi}_o\right]} \simeq \frac{Y_a(z,f) - U_o(f) - z\cdot\phi_o(f)}{\left[C_f - \frac{r_u^{(1)}}{r_c^{(1)}}\cdot\ddot{U}_o - \frac{r_\varphi^{(1)}}{r_c^{(1)}}\cdot\ddot{\phi}_o\right]} \tag{7}$$

$r_j^{(1)}$ being the participation coefficients for the first mode, relevant to the external force (r_c) and to the foundation displacement (r_u) and rocking (r_φ). The approximation introduced in eq. (7) can be easily accepted since in case of a structure such as a cantilever, the first vibration mode contributes almost totally to the global response in a frequency range close to the first resonance.

An example of the results of the analysis described above is shown in Fig. 5. The effects of soil interaction on the pier frequency is remarkable, especially for the pier founded on piles: for pier A the natural frequency of the actual pier is 2.48 Hz, whereas for "clamped" conditions rises to 2.82 Hz; for pier B is approx. 1.35 Hz and 1.46 Hz respectively. The shape of the curve (Fig. 5 a) shows that the pattern of the response of the system at the frequency of the "clamped" pier is close to that of a linear system. The very low value of the exciting forces at that frequency (Fig. 5 b), much lower than the force acting at the actual resonance frequency, explains such linear behaviour.

The knowledge of the natural frequencies of the "clamped" piers has also allowed an average value of the Young moduli for concrete to be computed: these resulted in $E_A \simeq 23000$ MPa for pier A and $E_B \simeq 28000$ MPa for pier B.

3.4. Piers behaviour

Since soil characteristics and effects have been found not responsible for the highly non-linear behaviour of the piers, attention has been then devoted to the response of the shaft, with particular reference to the strain and displacement meters installed across the cracks. The plots of the readings of such instruments versus time (see Fig. 6) generally show highly distorted patterns. Actually, when the acting moment makes the cracks open, the measured strains reach large values, whereas, when the moment reverses and the cracks close, the diagram suddenly flattens, indicating contact between the two edges of the crack.

The measured values of the width of the cracks shows that they cannot be attributed to local effects or surface shrinkage, so that they go deep into the cross-section of the shaft. A more significant representation of these results is given in Fig. 7, in which the crack opening is plotted versus the overturning moment: again the non-linear behaviour is clearly shown. In other cases, the diagram is almost linear (Fig. 8): in these cases however, the strains are always larger than those corresponding to a solid cross-section. This means that the reinforcement is mobilized to resist external forces even in compression, thus indicating that reinforcement has yielded, at least partially, or underwent some lack of adherence.

Of interest is also the diagram of Fig. 9, which illustrates a similar correlation, referring however to a strain meter placed in correspondence of the principal axis of the cross section perpendicular to the excitation directions: irrespective of the sense of the acting moment, the crack always opens, thus showing that the position of the neutral axis changes.

Further confirmation of the importance of the cracks is supplied by the plots of the maximum strains versus the overturning moment (Fig. 10): they again show that the strains are larger than those relevant to a solid cross-section (line b), although they are smaller than those corresponding to the mobilization of the reinforcement alone (line a).

4. CONCLUSIONS

The results of the experimental study illustrated above show that by a proper project of the tests, from the point of view both of the theoretical background for the interpretation of the results and data processing, and of the operating techniques, a number of information could be obtained which allowed the various aspects of the problem to be enlightened.

Soil-foundation effects have been determined, and have been found more important for the pier founded on piles; from the determination of the elastic and dissipative parameters of soil it has been concluded that its behaviour is linear. Consequently, the remarkable non linearity of the systems, which is not consistent with that of a monolithic and intact structure, has been attributed to the shaft.

The analysis of the shaft behaviour has confirmed such conclusion, with the determination of the importance and extent of the cracks.

A final remark is relevant to the intensity of the forces acting on the structures during the tests: the strains due to these forces at the base of the shaft were equal or greater to the dead load strains, and the dynamic loads on the soil ranged between 25% and 50% of the static ones.

Tests acceleration reached from 12% to 20% of the seismic accelerations produced by the EW component of the Irpinia earthquake recorded at Bagnoli, a few kilometers from the site of the bridge.

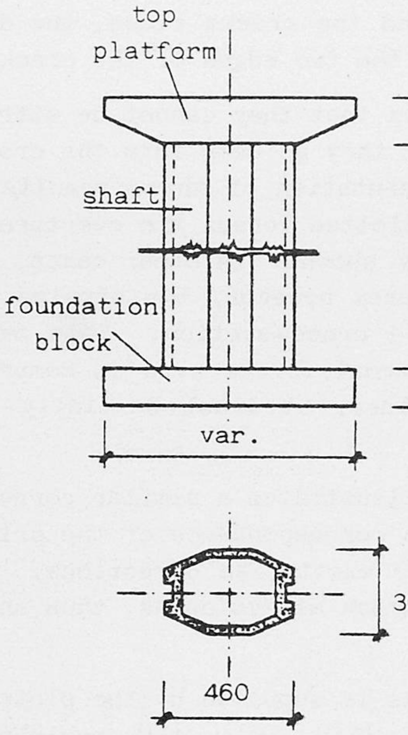

Fig. 1 Schematic representation of the pier geometry.

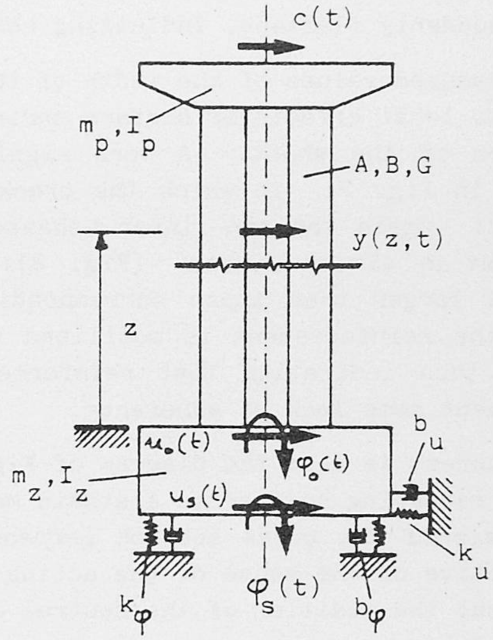

Fig. 2 Reference scheme for interpretation of soil behaviour.

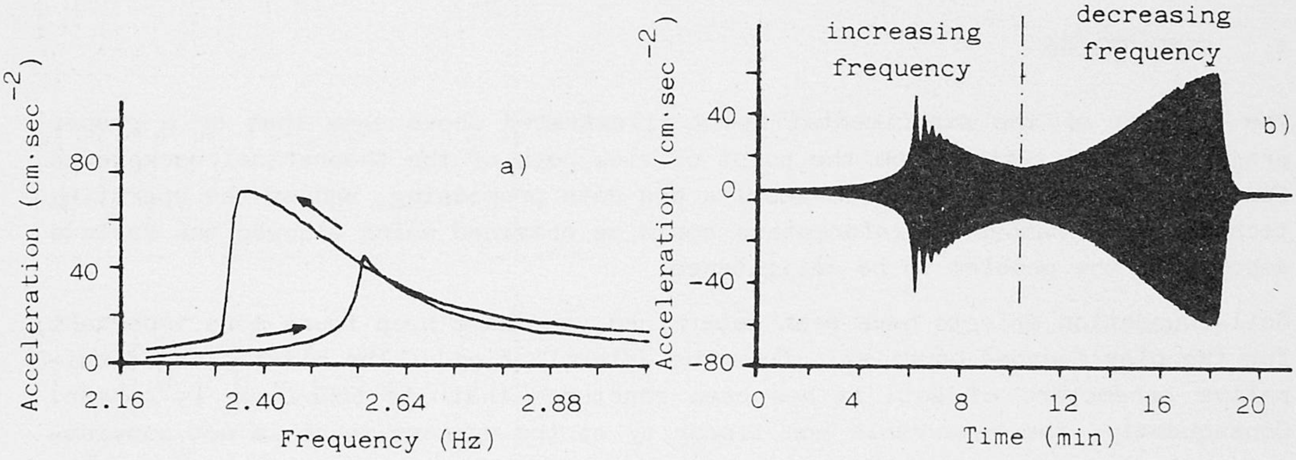

Fig. 3 Acceleration values at top of the piers. a) Response curves for pier A; b) Time history for pier B. Tests with increasing and decreasing frequency.

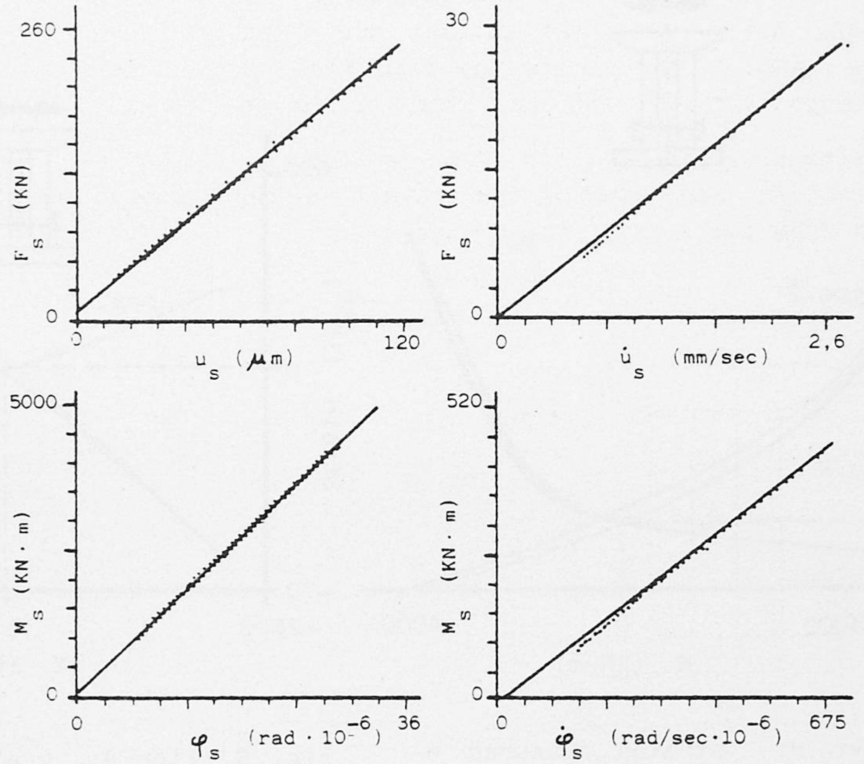

Fig. 4 Experimental correlation between applied loadings and foundation motions.

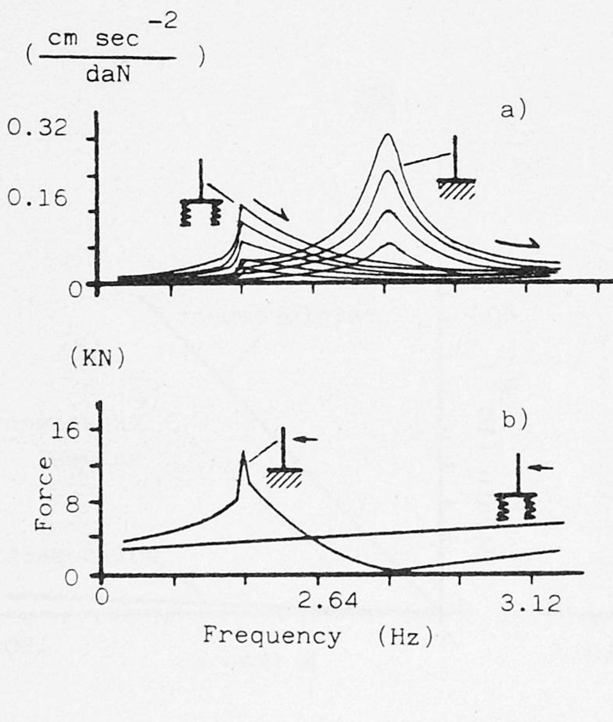

Fig. 5 Pier A: transfer functions (a) and exciting forces (b) for actual and "clamped" pier.

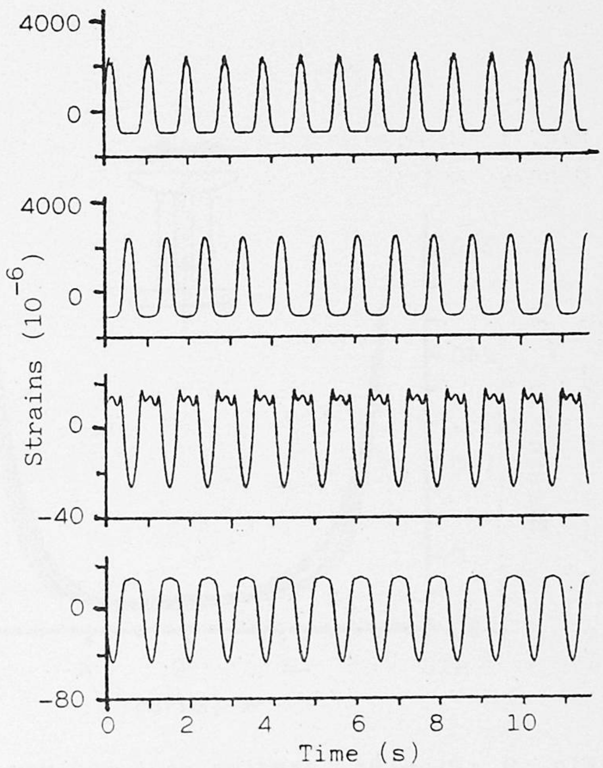

Fig. 6 Pier B: examples of strain behaviour showing contact of crack edges with shocks.

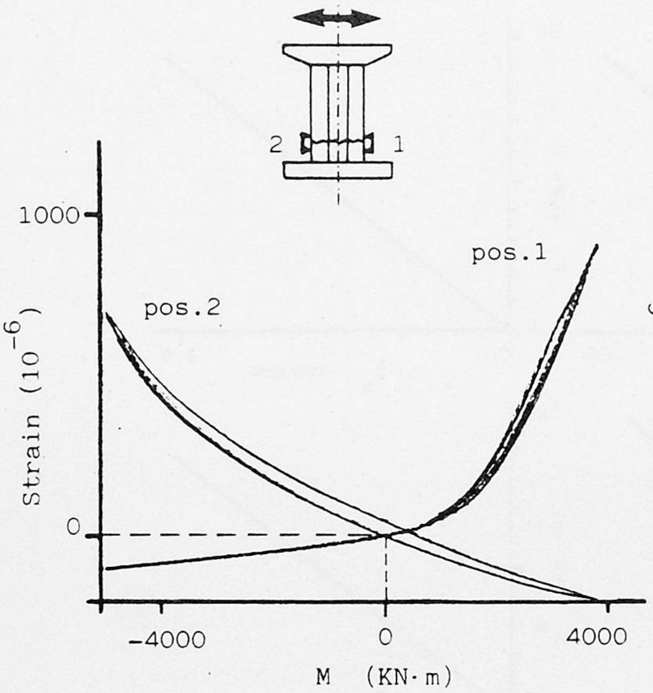

Fig. 7 Pier B: strains measured across cracks versus overturning moment. The non symmetric behaviour of the crack openings is cleary shown.

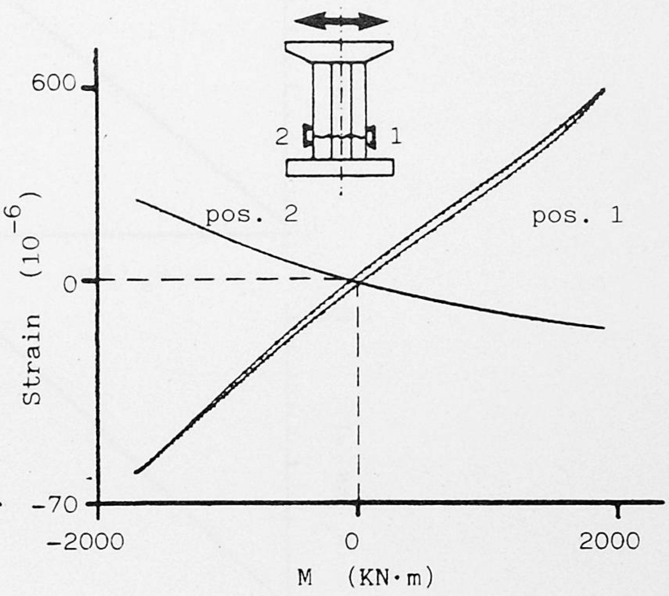

Fig. 8 Pier A: crack opening versus overturning moment. Position 1 shows mobilization of reinforcement in compression.

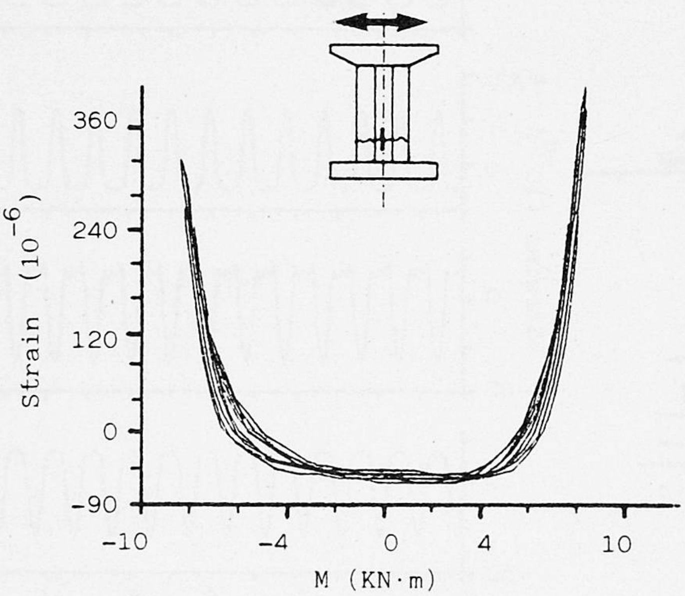

Fig. 9 Pier B: strains measured versus overturning moment across a crack on the "neutral axis" of the shaft.

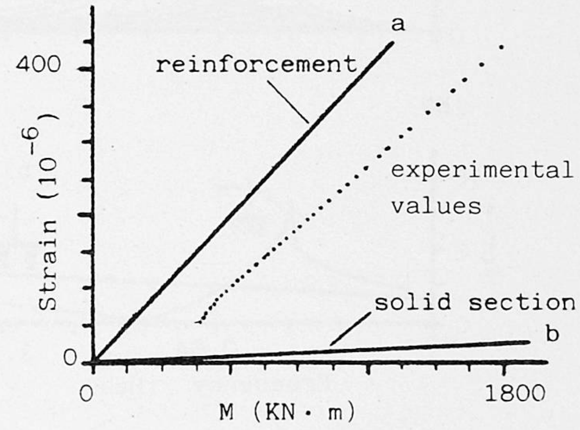

Fig. 10 Maximum strains versus overturning moment.