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POST-CRACKING BEHAVIOUR OF PIERS OF A WEIR OF A RUN-OF-RIVER POWER PLANT SUBJECTED TO EARTHQUAKE LOADING

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ABSTRACT

This paper presents a procedure for the seismic safety assessment of unreinforced concrete piers of a weir of a run-of-river power plant under earthquake loading. The nonlinear behaviour of the cracked pier was simulated by a presumed crack surface at which rocking of the upper portion of the pier can take place as soon as tensile stresses develop. The dynamic crack opening displacement was calculated by means of a time history analysis. Further, the likely earthquake damage has been predicted based on the probable maximum residual crack opening. Based on this study, it is concluded that the earthquake damage to the piers under consideration will be within the acceptable limits.

KEYWORDS

Pier; unreinforced concrete; rocking; post-cracking behaviour; damage.

INTRODUCTION

In order to judge the seismic safety of unreinforced concrete piers, it is important to know their post-cracking behaviour under earthquake loading. The results of such an investigation of the 80-year-old piers of the weir of the Augst-Wyhlen Hydro-Power Station (Fig. 1) on the Rhine River at the Swiss-German border are discussed in this paper. The power station, which has been under operation since 1912, is being renovated at present. There are a total of 11 concrete piers. Figure 2 shows a typical pier, which has a height of 20.9 m above the crest level of the barrage and has a stone facing. At the top, the piers support a steel bridge, on which there are various machines for the operation of the gates. A reinforced concrete bridge built by the well-known bridge engineer Maillart is located at the midheight of the piers on the downstream side.

EARTHQUAKE LOADING

The Augst-Wyhlen Hydro-Power Station is located approximately 20 km to the east of Basel, where a strong earthquake caused severe damage in the year 1356. The elastic design response spectrum with a maximum ground acceleration of 0.1 g specified in the Swiss code SIA 160 for stiff ground in zone 2 for a damping ratio of 5% was used as the earthquake loading. The average return period of this earthquake loading is about 400 years.

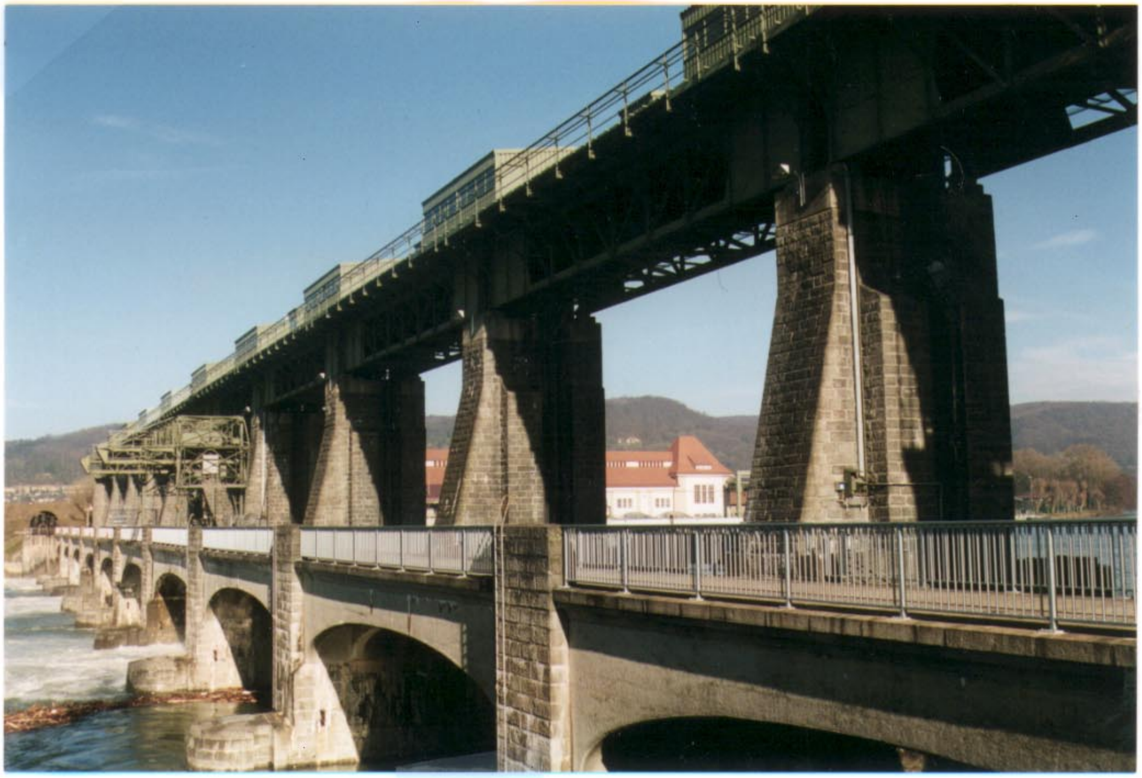

Fig. 1 Augst-Wyhlen Hydro-Power Station.

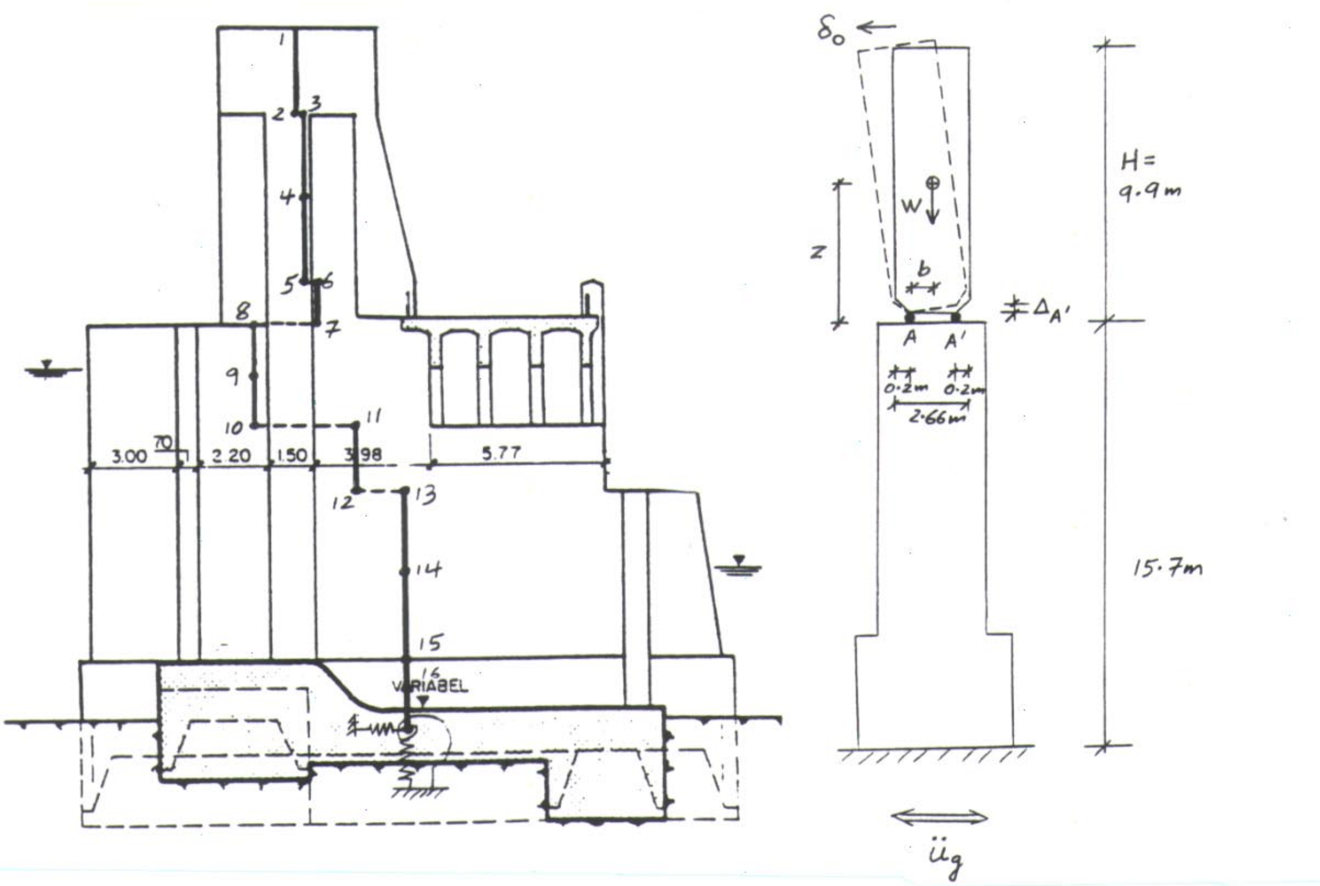

Fig. 2 Typical pier, idealized beam model and model with crack surface for rocking analysis.

For the rocking analysis of a pier in the post-cracking stage, artificial spectrum-compatible accelerograms were used. The 15 s long accelerograms were generated using the program SIMQKE (Gasparini and Vanmarcke, 1976) assuming 9 s of strong ground shaking. Because of the random nature of seismic ground motion, three different accelerograms were used. One of these accelerograms is shown in Fig. 3, together with the comparison of the achieved response spectrum with the target design spectrum.

LINEAR DYNAMIC ANALYSIS

For the linear response analysis, a three-dimensional beam model of the pier consisting of 17 nodes and 11 elements was used as shown in Fig. 2. The dynamic Young's modulus, the Poisson's ratio and the mass density were taken as 25 GPa, 0.2 and 2400 kg/m^3 for concrete, and 31.1 GPa, 0.3 and 2600 kg/m^3 for rock. The rock foundation was taken into account using simple lumped spring-mass-dashpot models for the six degrees of freedom as recommended in Newmark and Rosenblueth (1971). The eigenfrequency analysis shows that the first and second modes are in the cross-stream direction with frequencies of 4.5 Hz and 11.1 Hz. The lowest along-stream frequency is 12.5 Hz.

A linear dynamic analysis of the pier was performed using Rayleigh damping with damping ratios of 5% in the first two cross-stream modes. The maximum dynamic displacement and acceleration at the top of the pier calculated by the linear analysis are 4.2 mm and 0.32 g respectively. Similarly, the maximum dynamic tensile stress in the pier was calculated as 1.50 MPa at a level 9.9 m below the top of the pier where there is a sudden change of the cross-section. When the static stresses are added, the maximum tensile stress of 1.34 MPa occurs at the above-mentioned level. The old concrete in the pier is estimated to have only negligible tensile strength. Therefore, it can be expected to crack when subjected to strong ground shaking.

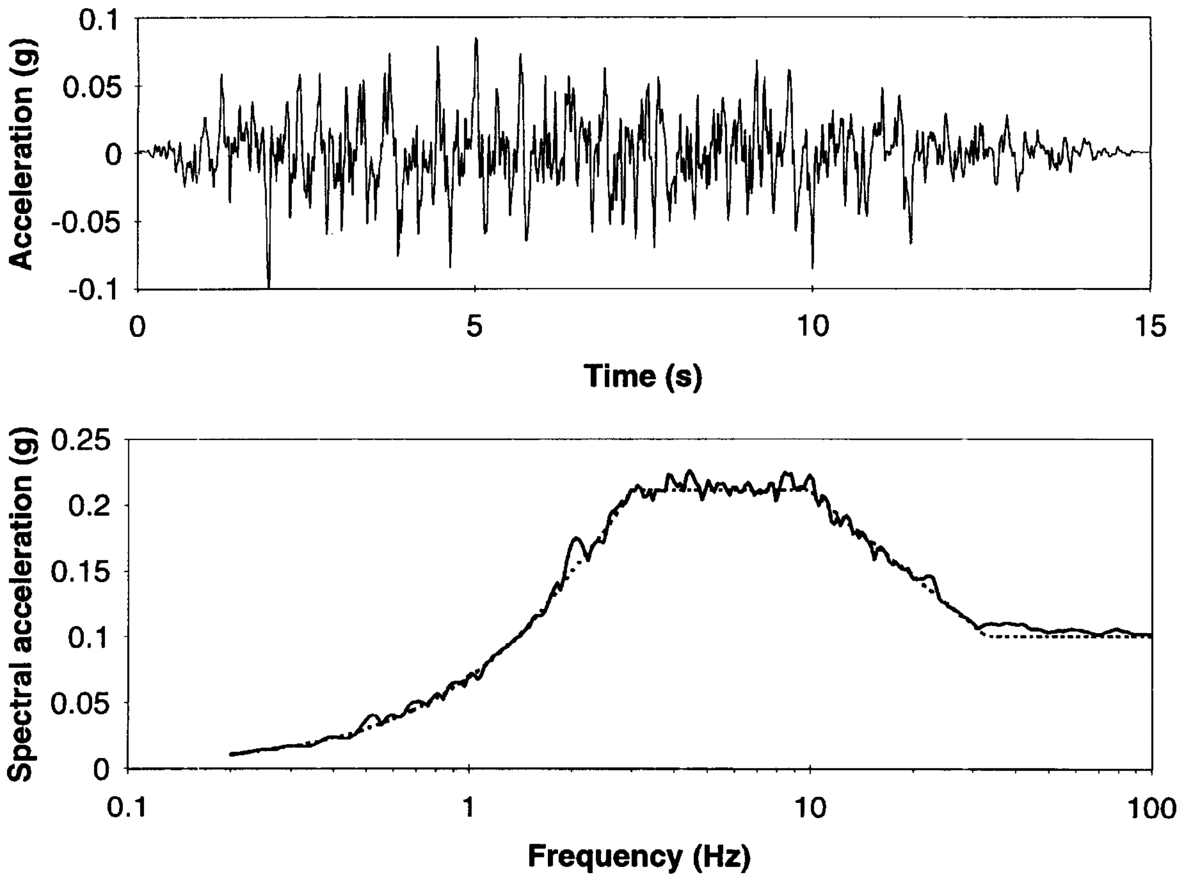

Fig. 3 Artificial spectrum-compatible accelerogram with peak ground acceleration of 0.1 g and comparison of target and achieved spectra.

NONLINEAR DYNAMIC ANALYSIS

In this study, a rocking analysis has been performed to investigate the post-cracking behaviour of the unreinforced concrete pier under seismic excitation. The problems of rocking and overturning of rigid blocks under earthquake action have been dealt with by many researchers. Some recent papers on this subject, where further references on related topics can be found, are those by Jones and Shenton (1990) and Winkler *et al.* (1995).

Post-Cracking Model

To study the post-cracking behaviour, a simple beam model of the cracked pier in the cross-stream direction was used as shown in Fig. 2. Since the pier length in the direction of the river flow is several times the width, bending stresses under the earthquake action are predominantly due to the horizontal component in the cross-stream direction, i.e., any rocking motion after the formation of a crack will also be in the same direction.

A horizontal crack surface is assumed to be present at the level where the maximum tensile stress was computed in the linear elastic analysis as discussed earlier. Energy absorption at the crack interface is disregarded. The nonlinearity of this presumed crack surface was modelled by using two nonlinear truss elements capable of taking only compressive forces as the contact between the upper and the lower portions of the cracked pier. Thus, rocking motion can occur in this model about either of the two axes at the edges of the crack surface as shown in Fig. 2. To take into account the possible spalling of the stone facing of the pier during the rocking motion, the rocking axes were taken 20 cm inside the faces of the pier.

Numerical Aspects

The rocking analysis was performed with the finite element program ADINA (ADINA R & D, 1987). For this analysis, only the horizontal component of the ground acceleration in the cross-stream direction was considered. The Newmark method was used for the time integration (Clough and Penzien, 1975).

Whereas a time integration step of 0.01 s was sufficient for a linear analysis, a very short integration step of 0.0001 s was found to be necessary for an accurate computation of the nonlinear behaviour. This is mainly because the abrupt reversal of the rocking motion involving the closure of the crack on one face and its opening on the opposite face causes high impact forces. As a result, very sharp high frequency peaks or spikes are present in the dynamic response.

DISCUSSION OF RESULTS

Three different nonlinear time history analyses were performed with three independent artificial accelerograms each with peak ground acceleration of 0.1 g. In the nonlinear time history analyses, the static loads were applied during the first two seconds, after which the ground acceleration was input.

Nonlinear Response and Floor Response Spectra

The maximum relative displacements of pier top are in the range of 4.5 mm to 4.8 mm. For comparison, the relative displacement computed through a linear elastic analysis, i.e. with no crack, is 4.2 mm. The peak relative acceleration of the pier top in the nonlinear model was calculated in the range of 0.23 g to 0.24 g, compared to 0.32 g if no crack was taken into account. The time history of the relative displacement at the pier top under the accelerogram of Fig. 3 is shown in Fig. 4. The envelope of the floor response spectra at the top of the cracked pier for 5% damping is shown in Fig. 5, where the elastic floor response spectrum (i.e. with uncracked pier) and the input SIA 160 spectrum are also shown.

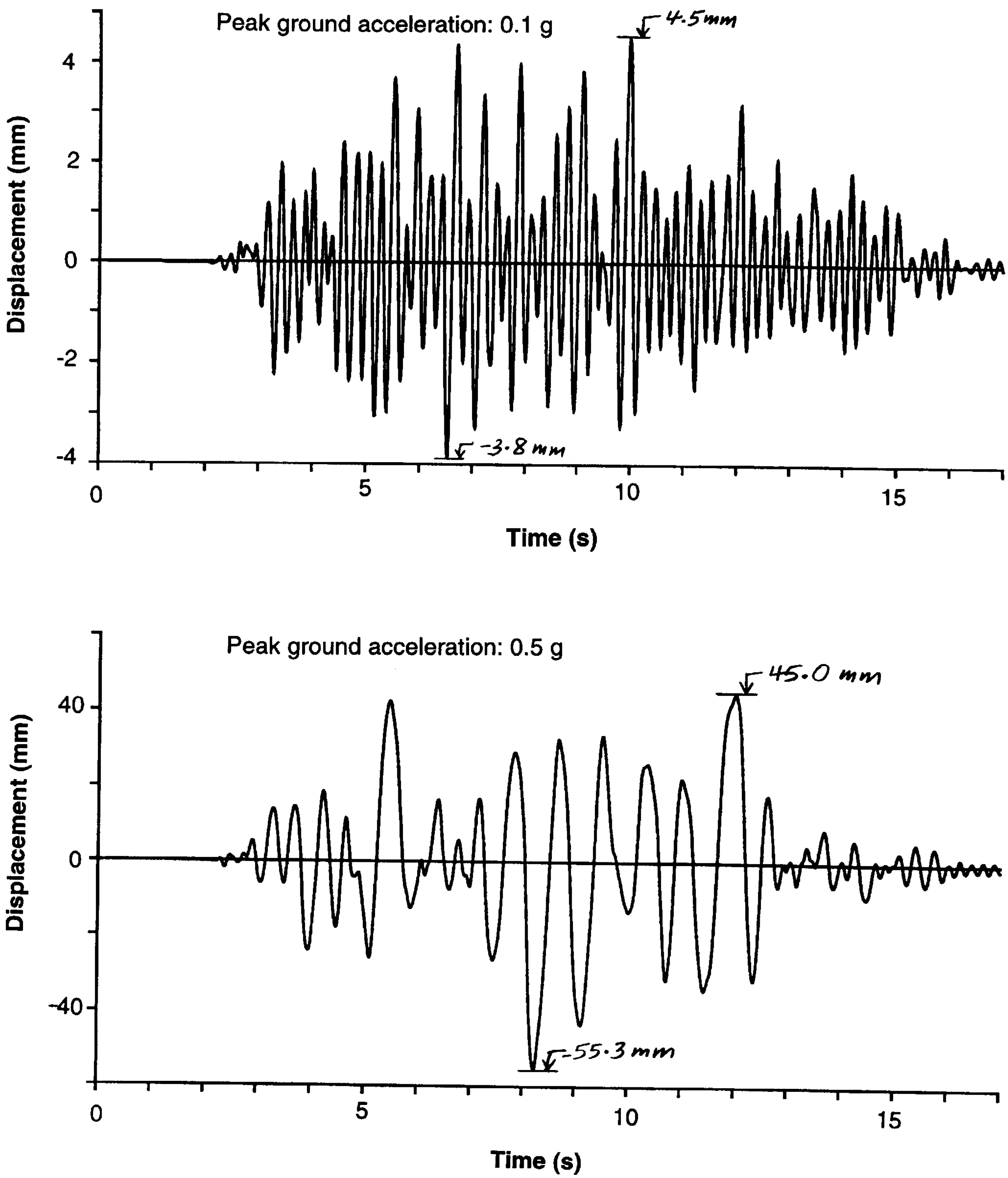

Fig. 4 Time histories of relative displacements at top of cracked pier under horizontal ground motion with peak accelerations of 0.1 g and 0.5 g acting in cross-stream direction.

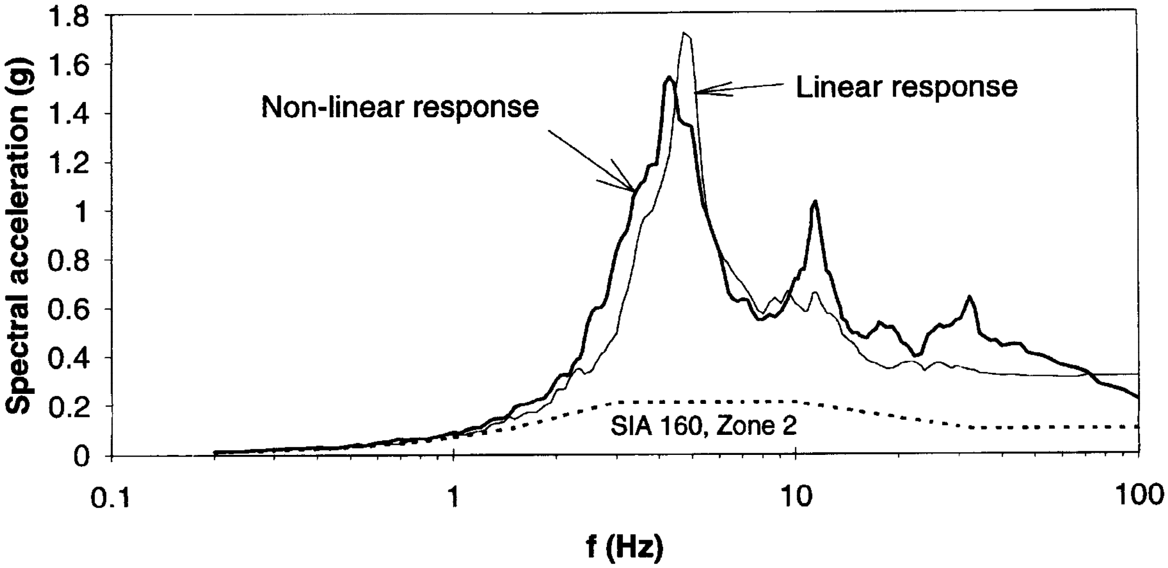

Fig. 5 Linear (thin line) and nonlinear (thick line) floor response spectra at pier top under ground motion with peak acceleration of 0.1 g acting in cross-stream direction; input SIA 160 spectrum is shown as dashed line.

Because of the crack formation, the dynamic bending moment at the crack surface cannot significantly exceed the static overturning moment at this level, which is equal to 4405 kNm in the present case. In the present analysis, the dynamic bending moment at this level is found to exceed the static overturning moment by about 11%. On the other hand, the linear elastic dynamic bending moment at the same level has a maximum value of 7994 kNm, which exceeds the dynamic bending moment calculated by the nonlinear analysis by 64%.

Rocking Frequency

The eigenfrequency f of rocking oscillations of a detached cantilever block having a weight W and a total height of H is a function of the maximum deflection δ_0 at the top and can be calculated from the following expression:

$$\cosh(\lambda / 4f) = (\beta / \lambda^2) / (\beta / \lambda^2 - \theta_0) \tag{1}$$

with $\theta_0 = \delta_0 / H$ (maximum rotation), $\beta = W b / I_0$ and $\lambda^2 = W z / I_0$, where z is the height of the centre of gravity of the block above the rocking axis, b is the horizontal distance between the rocking axis and the centre of gravity, and I_0 is the mass moment of interia of the block about the rocking axis. In the case of the piers under consideration, the natural rocking frequencies are calculated from eq. (1) as 2.36 Hz, 1.05 Hz, 0.73 Hz and 0.29 Hz for maximum pier top displacements of 1 cm, 5 cm, 10 cm and 50 cm, respectively.

For small values of δ_0, eq. (1) can be changed to the simple form

$$f = \sqrt{W b H / (32 I_0 \delta_0)} \tag{2}$$

Equation (2) shows that the natural frequency of rocking oscillations of a rigid block with small amplitudes is inversely proportional to the square root of the maximum rocking displacement.

Crack Opening

The time history of the crack opening at one of the two edges of the crack surface under the accelerogram of Fig. 3 is shown in Fig. 6. The maximum crack openings due to all three accelerograms having a peak value of 0.1 g were about 0.9 mm.

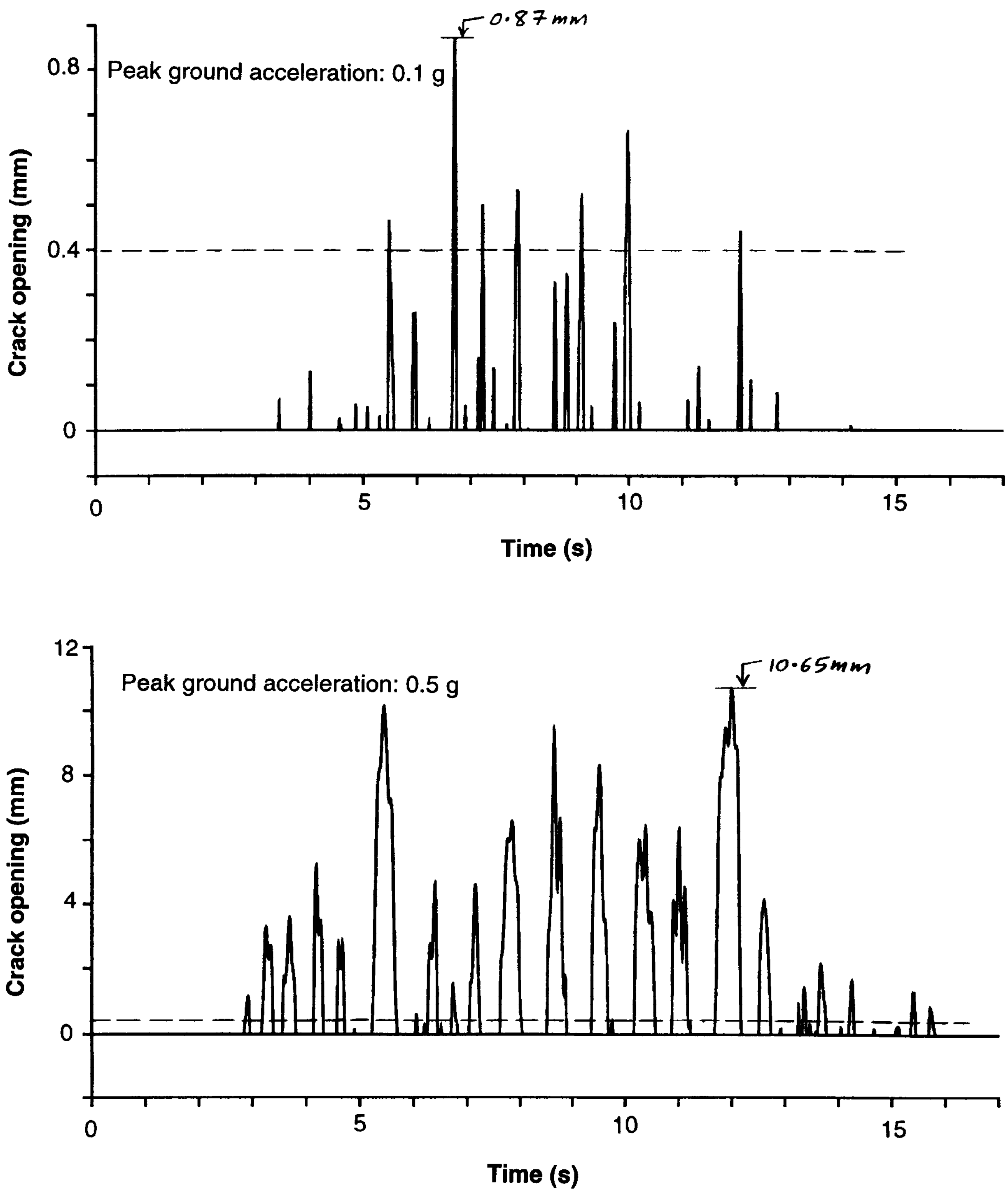

Fig. 6 Time histories of crack openings at one of two edges of crack surface under horizontal ground motion with peak accelerations of 0.1 g and 0.5 g acting in cross-stream direction.

To estimate the damage due to the earthquake action, the residual crack opening was calculated by accumulating all crack openings exceeding 0.4 mm. This has been done under the arbitrary assumption that any crack opening exceeding 0.4 mm is irreversible. The residual crack opening after the earthquake was estimated in the range of 1.3 mm to 4 mm with the three accelerograms considered, with the corresponding permanent horizontal displacement of 5.7 mm to 17.5 mm at the pier top.

Effects of Extreme Seismic Events

In order to study the effects of ground accelerations exceeding the design ground acceleration, additional calculations were carried out using accelerograms with peak ground accelerations of 0.2 g, 0.3 g and 0.5 g. The maximum dynamic crack openings were calculated as 3.6 mm, 5.7 mm and 13.5 mm respectively for these three cases. The residual crack openings are respectively equal to 31 mm, 51 mm and 104 mm, with the corresponding maximum displacements of the pier top in the cross-stream direction equal to 136 mm, 223 mm and 456 mm, respectively.

The time histories of the horizontal displacement at the pier top and the crack opening displacement at one crack surface edge under the seismic action with the peak ground acceleration of 0.5 g are shown in Figs. 4 and 6. From Fig. 4, it is obvious that the rocking frequency is reduced when the amplitude increases in a manner already discussed. Each crack opening under a stronger ground excitation lasts for a longer duration, as is clear from Fig. 6.

CONCLUSIONS

Based on the results of this inelastic analysis, it is concluded that the earthquake damage to the piers will be limited. The maximum dynamic crack opening of 0.9 mm and the estimated permanent horizontal displacement of up to 17.5 mm at the pier top can be accepted. Therefore, no special strengthening measures are required. However, the sliding stability of the piers are to be improved by modifying the foundations of the piers and the weir, and by installation of rock anchors.

At the pier top, peak horizontal accelerations of up to 0.24 g will occur during an earthquake. The steel bridge supported on the pier top and the electro-mechanical equipment on it must be able to withstand this level of excitation to ensure the safe operation of the power plant after an earthquake.

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