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Shake table experiments for cracking and sliding response of concrete dams

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ABSTRACT

This paper presents shake table experiments on four 3.4 m high plain concrete gravity dam models to investigate their dynamic cracking and sliding responses. The experimental results are then compared with a Finite Element nonlinear constitutive smeared crack model based on fracture mechanics principles. For the sliding mechanism, the numerical simulations are based on rigid body dynamics with frictional strength derived from the Mohr-Coulomb criterion. Very good correlation between the observed experimental cracking and sliding responses and numerical simulations is obtained.

INTRODUCTION

The seismic safety of existing concrete dams is a major concern, due to the catastrophic consequences of a sudden release of the reservoir, if the dam fails under strong ground motions. Most existing dams were built many years ago with minimal consideration for seismic loads. The state of knowledge in structural dynamics and seismicity is continuously progressing such that the seismic safety of concrete dams should be periodically evaluated considering the latest assessment of their strengths and the ground motion intensity to which they might be subjected.

Guidelines for seismic evaluation of dams have been adopted in several countries including Canada (CDSA 1995) in addition to those suggested by the International Commission on Large Dams. These guidelines state that, under an annual exceedance probability of 10^{-3} to 10^{-4} for a maximum design earthquake (MDE), the dam should not slide, open at joints, or crack to the extent that uncontrolled leakage would take place, nor should the blocks of the upper part be significantly displaced. Earthquakes have damaged several large concrete dams during this century without release of the reservoir. This good performance is largely attributed to the fact that a very small number of concrete dams have been shaken by a severe earthquake of duration long enough to induce significant damage or failure. Thus the majority of dams have never been tested for intense earthquakes with current ground motion criteria. In addition to historical evidences, there are also a number of experimental shake table tests that were conducted on small scale models to predict the earthquake response of gravity dams. These experiments have been reported elsewhere (Cipolla 1998).

On the numerical side, two different approaches have been used to advance this field. Using the discrete crack approach and fracture mechanics, remeshing of the finite element model is required at each time increment. For the smeared crack approach, the finite element mesh is fixed and the isotropic concrete constitutive relations are replaced with orthotropic properties. To improve mesh objectivity for the nonlinear solution, the smeared crack or the damage mechanics approaches introduce the tensile softening modulus related to the fracture energy of concrete, G_f, and the characteristic length of the elements (CEA 1998, Tinawi et al. 1999).

From the compiled historical, experimental and numerical evidences related to seismic safety of concrete gravity dams, the objectives of this paper are:

- To perform shake table tests to determine the critical acceleration for which cracking in two monolithic dam models is observed, and to correlate the experimental cracking response with a G_f-type smeared crack model (Bhattacharjee and Léger 1993);

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- To obtain experimentally the sliding response for two other dam models with the same height, but having an unbonded lift joint at about mid-height, and to correlate the observed cumulative displacements, due to sliding, with the results obtained from a time integration algorithm, using rigid body dynamics.

DYNAMIC CRACKING RESPONSE

The design of the specimens, satisfying experimental limitations resulted in a 3.4 m dam model with a block crest of 0.5 m by 0.5 m and a downstream slope of 0.7. The thickness of the specimen was 0.25 m as shown if Fig. 1. These dimensions for the dam model yielded a natural frequency of 80Hz, which is beyond the operational range of the table. To reduce the natural frequency to about 30 Hz, a simple solution was to anchor a 2700 kg mass at the crest. To control the location of crack initiation, two notches 10 mm thick and 250 mm long were introduced on the upstream and downstream faces of the model. These notches, located 1m above the footing, decreased the natural frequency of the specimens to approximately 20Hz. The total mass of the specimen, including the added mass at the crest, is 6800 kg. The specimens were instrumented with five accelerometers and three wire transducers to measure displacements.

A fixed base FE model has resulted in a first natural frequency of 21.8 Hz for the monolithic specimen, while laboratory impact tests showed a lower value of 16.4 Hz for the first specimen, and 17.0 Hz for the second one. It was suspected that the shake table was influencing the dynamic properties of the experimental setup. To take into account the influence of the shake table, a uniform distribution of vertical springs was introduced at the base of the FE model (Fig. 2). The stiffness of the springs in the FE model was adjusted to duplicate only the first natural frequency. Viscous damping for the monolithic specimens was measured using the free vibration decay technique. This resulted in a value of 0.9% for the first specimen and 1.3% for the second specimen.

The two monolithic specimens were subjected to simple acceleration pulses to observe their cracking responses. Simple pulses are easier to use and interpret compared to seismic records. The total duration of the pulse is 0.1 sec and is made up of two triangular acceleration segments, one positive and the other negative. The first monolithic specimen was therefore subjected to this triangular acceleration pulses. The first peak acceleration (FPA) was increased progressively in approximate increments of 0.1g. Up to an FPA level of 0.87g no cracking was observed as shown in Fig. 3(a). By increasing further the acceleration level, a partial crack was observed at a FPA = 0.94g (Fig. 3(b)). This partial crack initiated on the downstream face, at the notch level, with an initial angle of about 30° downwards and propagated, at an average velocity of 35 m/s, about 350 mm as shown in Fig. 3(b). By attempting to repeat the last acceleration level input, the crack extended fully in a near horizontal plane, to reach the other notch on the upstream face (Fig. 3(c)) at a FPA= 0.98g.

The second homogeneous specimen was tested with the objective to determine the influence of subsequent pulses, over the first, on the crack propagation. It was therefore decided to increase the number of applied pulses from one to three. From Fig. 3(d) it is observed that three consecutive pulses with a FPA of 0.96g were sufficient to fully crack the dam into two separate bodies. Laboratory cracking shows slightly curved profiles at initiation, while numerical simulations show straight horizontal profiles. In fact, the numerical simulations do predict the initial inclination of the crack (30°) which is not sufficient to drive the crack downwards. As the cracking profile progresses, the principal stress orientation tends to decrease, consequently favouring a linear horizontal crack propagation in the numerical simulations. The overall cracking orientation, observed in the laboratory, is well predicted by the numerical simulations using FRAC-DAM (Bhattacharjee 1996). The fracture energy of the concrete used was 60 N.m/m² for the first specimen and 54 N.m/m² for the second specimen. A dynamic amplification factor of 1.75 was used for both the fracture energy (G_f) and for the static tensile strength (f_t=1.6MPa). This dynamic amplification was obtained by comparing the numerical results with the test where no cracking was observed (Fig. 3a). An amplification factor of 1.25 was used for the static concrete elastic modulus (E=14.9 GPa) to account for the dynamic nature of the load.

DYNAMIC SLIDING RESPONSE

Two instrumented specimens with a cold lift joint, introduced 1m above the footing, by casting the lower part of the specimen separately from the upper part, were constructed as shown in Fig. 4. The main objective of these experiments is to study the sliding behavior under quasi-static and dynamic loads. To avoid rocking of the upper block, the 2700 kg crest mass used earlier, during the cracking tests, was removed. However, to induce downstream sliding, a horizontal force is introduced with a 700 kg hanging mass attached to the upstream face of the specimen with a cable.

To break the bond at the joint location, a 10 Hz sinusoidal acceleration, with increasing amplitude, was used for the first specimen. Failure and subsequent sliding occurred at 2.1g. For the second specimen, failure of the joint was obtained using the Chicoutimi North transverse record (PGA=0.13g) of the 1988 Magnitude (M_s) 5.7 Saguenay earthquake. Failure and subsequent sliding occurred when the PGA was increased to a value of 0.5 g. More details have been reported by Cipolla (1998).

The forces acting on the upper block can be cast in the following equation of motion:

$$\ddot{s}(t) = -a(t) - \mu g + \frac{F(t)}{M} \tag{1}$$

where M is the 1300 kg mass of the upper block, $\ddot{s}$(t) is the sliding acceleration of the upper block, a(t) is the input table acceleration, F(t) is the cable tensile force, and μ is the friction coefficient. The measured variation in cable force (F(t)) is considered in the integration of the equation of motion for the sliding response. Static tests, on both specimens, to obtain the coefficient of friction yield a value for μ = 0.68 for the peak response, and μ = 0.6 for the residual response, corresponding to the basic friction coefficient. The critical acceleration must be overcome for sliding to occur. The value of the critical acceleration, a_{cr}, is obtained from equation (1) when $\ddot{s}$(t) is zero. In general, when there is no cable force, the critical acceleration, a_{cr}/g, is equal to μ. However, this value is reduced due to the presence of the cable force. If F(t) is assumed constant, the downstream critical acceleration is 0.0615g. The upstream critical acceleration is 1.14g.

Figure 5 shows the sliding response of the second specimen with a joint, submitted to the 1988 Saguenay record scaled to a PGA = 0.53g, and the 1940 El Centro (S00E component) record scaled to 0.28 g. The numerical simulation using the Saguenay record correlated well with the experimental test using a constant value (μ = 0.6) for the friction coefficient. El Centro record overestimated the experimental sliding response, when using μ = 0.6. In this case, non-negligible rocking rotations (around 0.1°) occurred during the experimentation and a much better correlation is obtained using μ = 0.64. The Saguenay record with a PGA increased to 0.53g produced experimentally sliding displacements of the order of 93 mm only. The El Centro earthquake with a PGA scaled down to 0.28g produced experimental sliding response of the same order (123 mm). Therefore, the PGA is not the only indicator for estimating the sliding response. The frequency content and the duration of a seismic record must be considered.

CONCLUSIONS

Shake table tests have been carried out on four 3.4 m high concrete dam models. Two specimens were monolithic and provided information on the cracking response when subjected to simple triangular acceleration pulses. The other two specimens had cold joints that were allowed to slide after breaking them. Sliding tests included North American eastern (high frequency) and western (low frequency) acceleration records. The tests provided an opportunity to investigate the robustness, or to highlight deficiencies, of existing numerical procedures.

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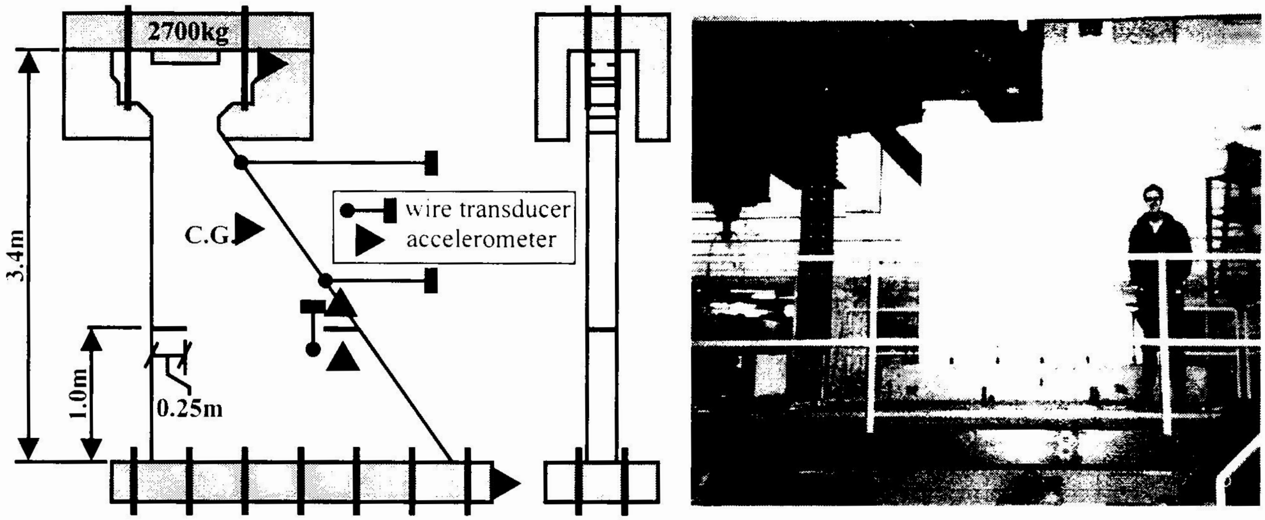

Figure 1: Monolithic dam model and instrumentation setup.

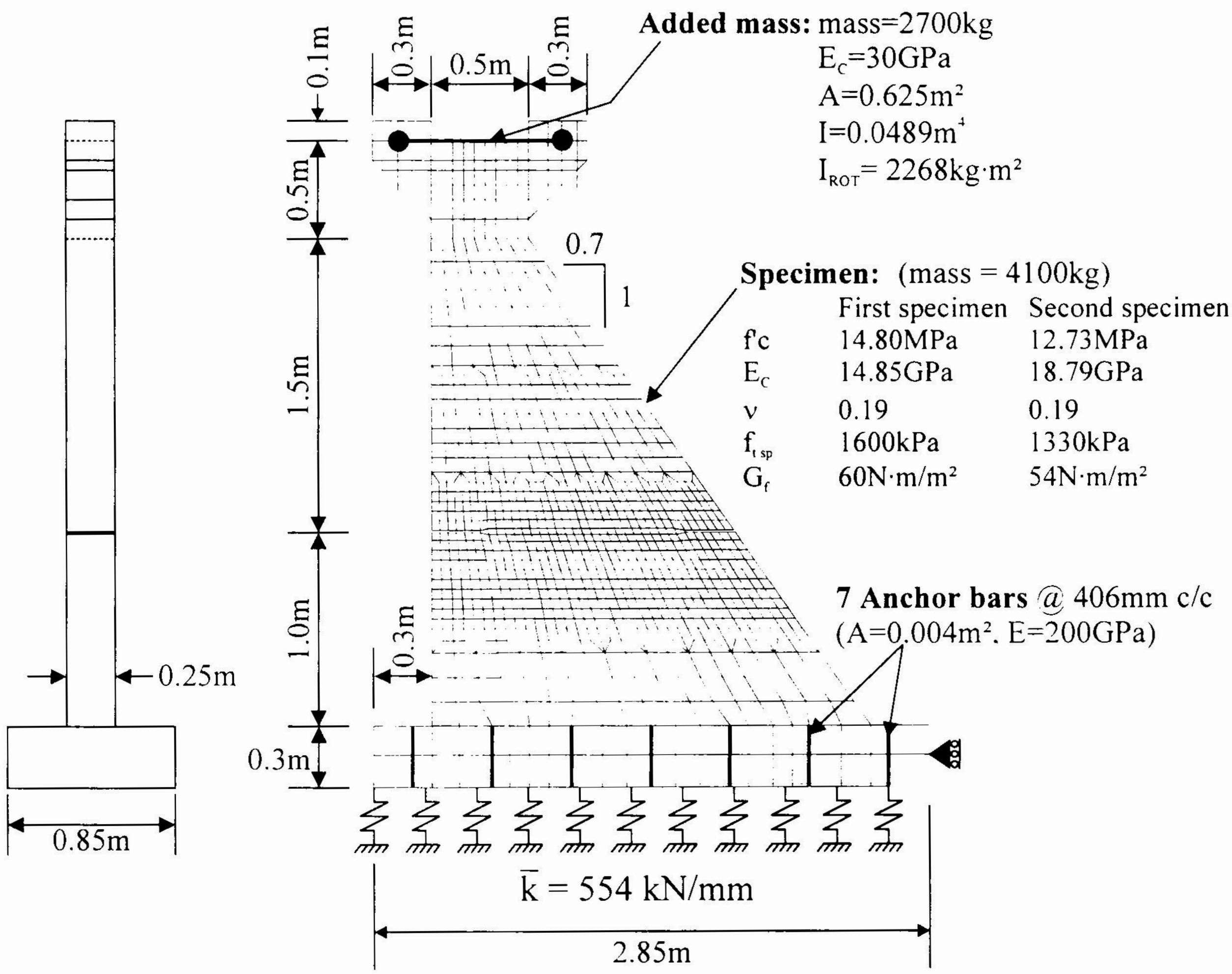

Figure 2: Finite element model.

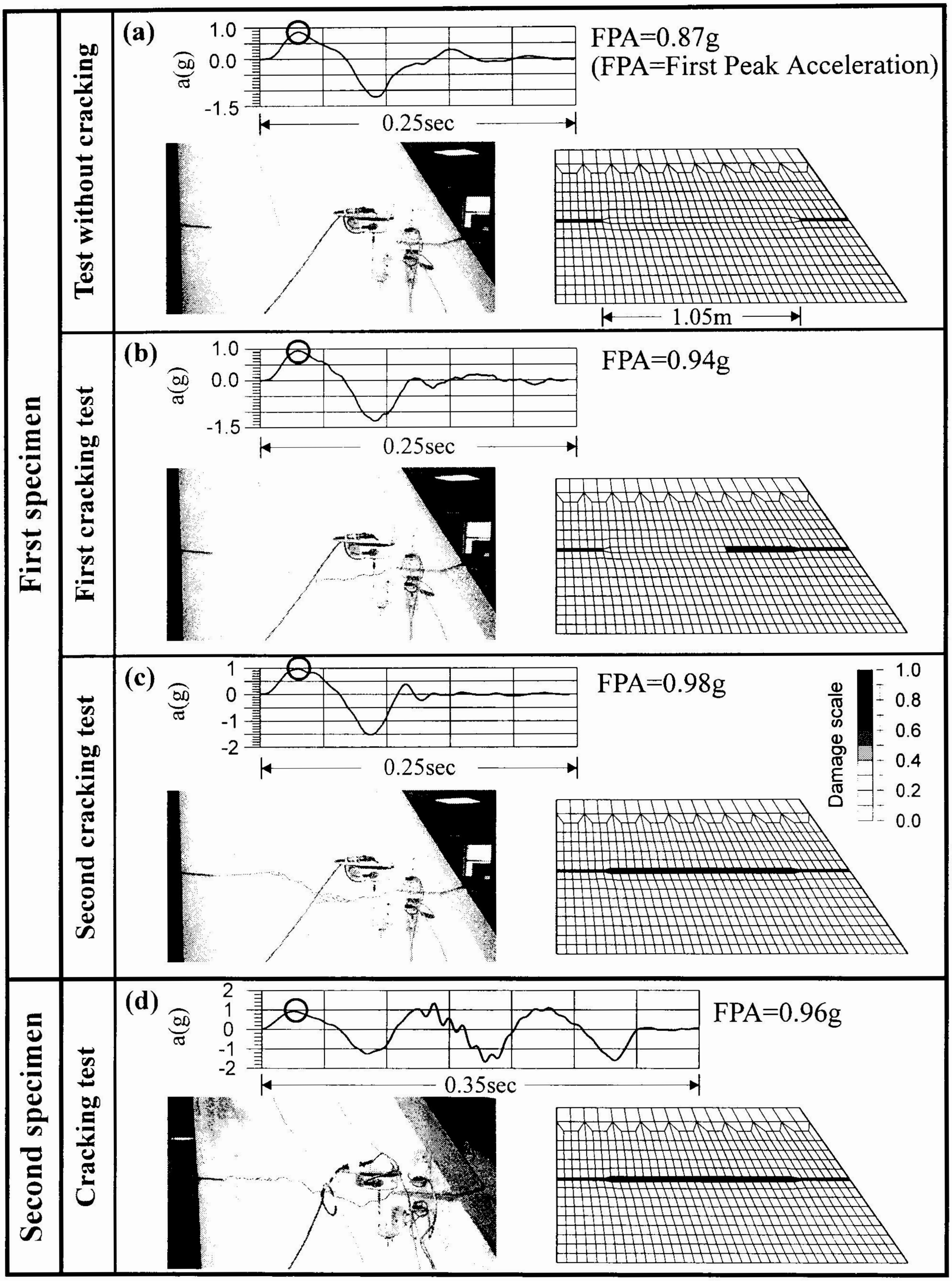

Figure 3: Observed cracking at the lab and related numerical simulations.

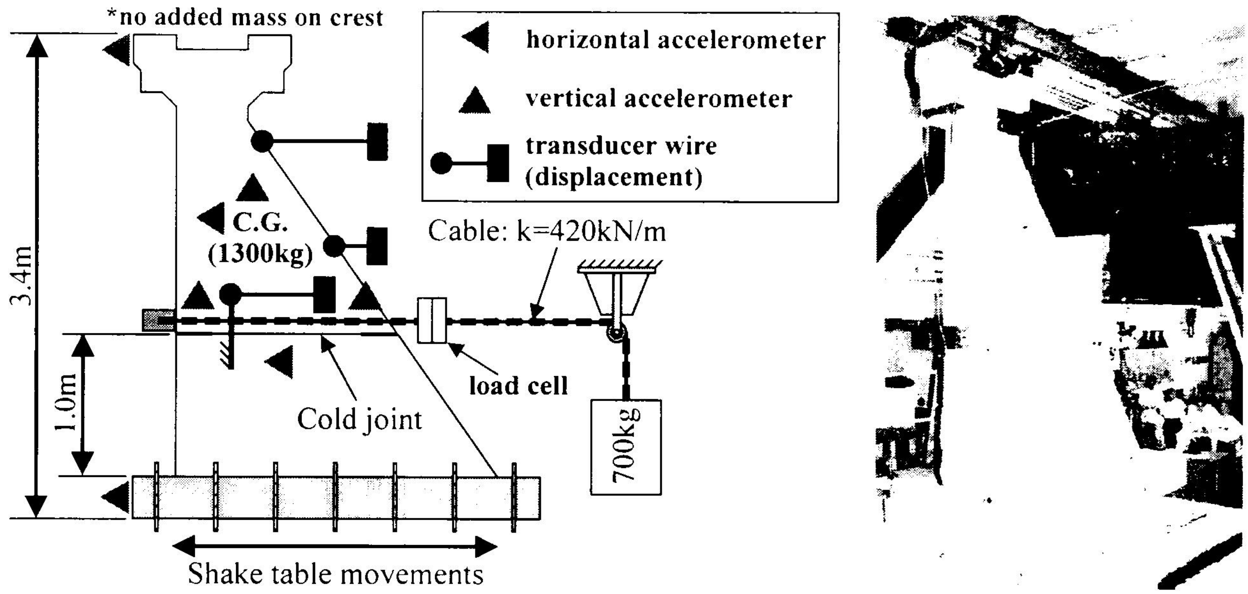

Figure 4: Instrumentation and experimental setup for the dam model with a cold joint.

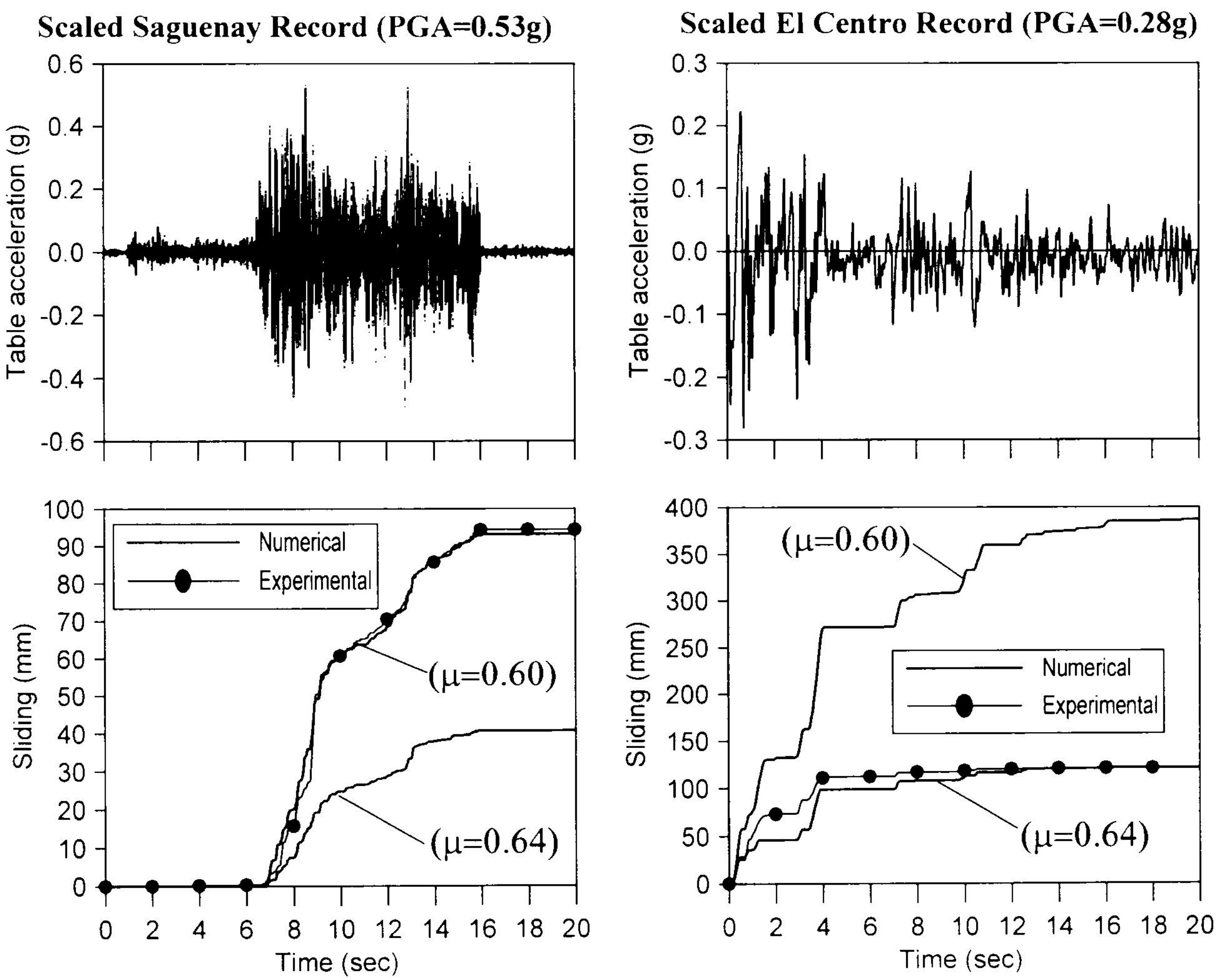

Figure 5: Sliding responses of the second specimen with a joint submitted to the Saguenay and El Centro records.