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# Centrifugal modelling and analysis of soil-structure interaction

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ABSTRACT: This paper describes a centrifuge test in which a heavy structure, embedded to about one-quarter of its height in a saturated sand foundation, was subjected to simulated earthquake motions. Accelerations and porewater pressures were measured at several points in the foundation soil and both horizontal and vertical accelerations were measured on the structure itself. The soil-structure system was analyzed using the nonlinear dynamic analysis program TARA-3. A comparison between measured and computed responses shows that the nonlinear program gives very good estimates of both horizontal and vertical accelerations and passes all the frequencies of interest, including translational and rocking frequencies, from the foundation into the structure. Reliable estimates of porewater pressures are also obtained.

## 1. INTRODUCTION

Dynamic soil-structure interaction during earthquakes is a very complex phenomenon because of the nonlinear response of soil to strong shaking. The interaction becomes even more complex if the soil is saturated and large seismically induced porewater pressures are generated which alter the strength and stiffness of the soil. In current engineering practice, soil-structure interaction effects are usually analyzed using equivalent linear finite element analyses in the frequency domain. There are few field data to validate how well these methods work.

Certain important phenomena in soil-structure interaction are outside the scope of conventional frequency domain analysis. Typical examples are uplift during rocking, stick-slip behaviour at interfaces, permanent deformations, the effects of increasing porewater pressures and hysteretic behaviour. To model these phenomena, nonlinear dynamic effective stress analysis in the time domain is necessary.

Siddharthan and Finn (1982) developed a dynamic nonlinear effective stress method of analysis which they incorporated in the computer program TARA-2. A modified and extended version of this program called TARA-3 was developed by Finn et al. (1986a) which has a number of additional options such as energy transmitting boundaries and the facility of computing nonlinear consolidation during and after seismic excitation. A very brief description of TARA-3 is presented later.

The United States Nuclear Regulatory Commission (NRC) through the U.S. Army Corps of Engineers sponsored a series of centrifugal model tests to provide data for the verification of TARA-3. The tests were conducted on the large geotechnical centrifuge at Cambridge University in the United Kingdom. The models were surface and embedded structures with foundations of both dry and saturated sands. Some of these tests have been described previously by Finn (1985) and Finn et al. (1984, 1985a, 1985b, 1986b). One test will be described here which models the response of a heavy two-dimensional structure embedded in a saturated sand foundation to seismic excitation.

The response of the model was computed using TARA-3 and a comparison of computed and measured responses is presented.

## 2. CENTRIFUGE MODEL TEST

Details of the Cambridge geotechnical centrifuge and associated procedures for simulated earthquake testing have been described by Schofield (1981). Seismic excitation of centrifuged models in the

Cambridge centrifuge is generated by a wheel travelling on a track with precisely machined undulations attached to the wall of the centrifuge pit and extending over one-third of the circumference. The system is referred to as the Bumpy Road. The intensity of model shaking is controlled by adjusting the linkage between wheel and model container. For a given Bumpy Road configuration, frequency of oscillation is governed by the angular velocity of the rotor arm.

Ideally, the Bumpy Road should generate sinusoidal pulses but resonances, mechanical linkage clearances and other factors result in some high frequency signals which broaden the frequency input range. In particular, the model earthquake consists of three distinct phases:

(1) small "wheel-on" accelerations associated with initial contact of the wheel with the track;
(2) the model earthquake proper consisting of roughly sinusoidal pulses;
(3) small "wheel-off" accelerations associated with the wheel leaving the track.

The base input motion is transmitted through the model primarily by shear stresses but also by normal stresses due to bending modes. However, some high frequency acceleration spikes are introduced due to the effects of the sides of the container and container resonances.

A schematic view of the model structure is shown in Figure 1. It is made from a solid piece of aluminum alloy and has dimensions 150mm wide by 108mm high in the plane of shaking. The length perpendicular to the plane of shaking is 470mm and spans the width of the model container. The structure is embedded a depth of 25mm in the sand foundation. Sand was glued to the base of the structure to prevent slip between structure and sand.

The foundation was constructed of Leighton Buzzard Sand passing British Standard Sieve (BSS) No. 52 and retained on BSS No. 100. The mean grain size is therefore 0.225mm. The sand was placed as uniformly as possible to a nominal relative density $D_r = 52\%$.

De-aired silicon oil was used as a pore fluid in order to model the drainage conditions in the prototype during the earthquake. In a 1/N linear scale model, excess porewater pressures dissipate $N^2$ times faster in the model than in the prototype if the same fluid is used in both. The rate of loading by seismic excitation will be only N times faster. Therefore, to model prototype drainage conditions during the earthquake a pore fluid with a viscosity N times the prototype viscosity must be used. This viscosity was achieved by an appropriate blending of commercial silicon oils. Tests by Eyton (1982) showed that the stress-strain behaviour of fine sand was not changed when silicon oil was substituted for water as a pore fluid.

During the test the model experienced a nominal centrifugal acceleration of 80 g. The model therefore simulated a structure approximately 8m high by 12m wide embedded 2m in the foundation sand.

The horizontal and vertical accelerations were recorded by DJB A23 piezoelectric accelerometers (ACC); the porewater pressures were recorded by Druck PDCK 81 porewater pressures transducers (PPT). The locations of the accelero-

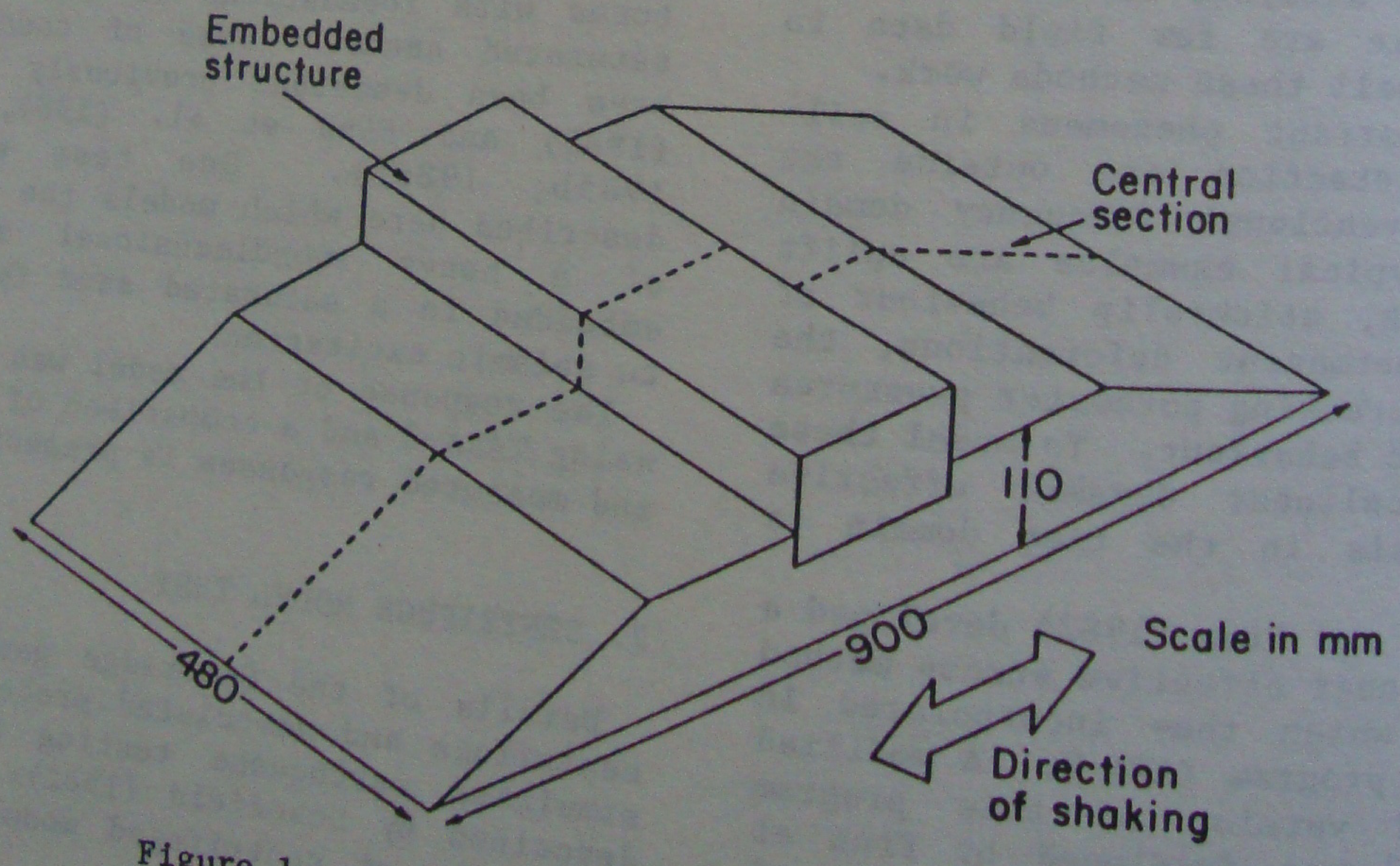


Figure 1. Centrifugal Model of Embedded Structure.

meters and pressure transducers from which data will be reported are shown in Figure 2. Since interest is primarily in whether TARA-3 can model interaction effects, the accelerations are reported only for accelerometers mounted on the structure except for ACC 3441 which recorded the input motions (Figure 3). Horizontal accelerations at the top of the structure were recorded by ACC 1938; ACC 1900 and ACC 1572 recorded the vertical accelerations due to rocking.

Porewater pressures recorded by PPT 2851 in the free-field, PPT 2846 near the edge of the structure and PPT 2631 under the structure illustrate the effects of soil-structure interaction on porewater pressures.

## 3. TARA-3 METHOD OF ANALYSIS

An incrementally elastic approach was adopted in TARA-3 to model nonlinear behaviour using tangent shear and bulk moduli, $G_t$ and $B_t$ respectively. The incremental dynamic equilibrium forces $\{\Delta P\}$ are given by

$$[M]\{\Delta\ddot{x}\}+[C]\{\Delta\dot{x}\}+[K]\{\Delta x\} = \{\Delta P\} \qquad (1)$$

where [M], [C] and [K] are the mass, damping and stiffness matrices respectively, and $\{\Delta x\}$, $\{\Delta\dot{x}\}$ and $\{\Delta\ddot{x}\}$ are the matrices of incremental relative displacements, velocities and accelerations. The viscous damping is of the Rayleigh type and the stiffness matrix is a function of the current tangent moduli. The use of shear and bulk moduli allows the elasticity matrix [D] to be expressed as

$$[D] = B_t[Q_1] + G_t[Q_2] \qquad (2)$$

in which $[Q_1]$ and $[Q_2]$ are constant matrices for the plane strain conditions usually considered in analysis. This formulation reduces the execution time

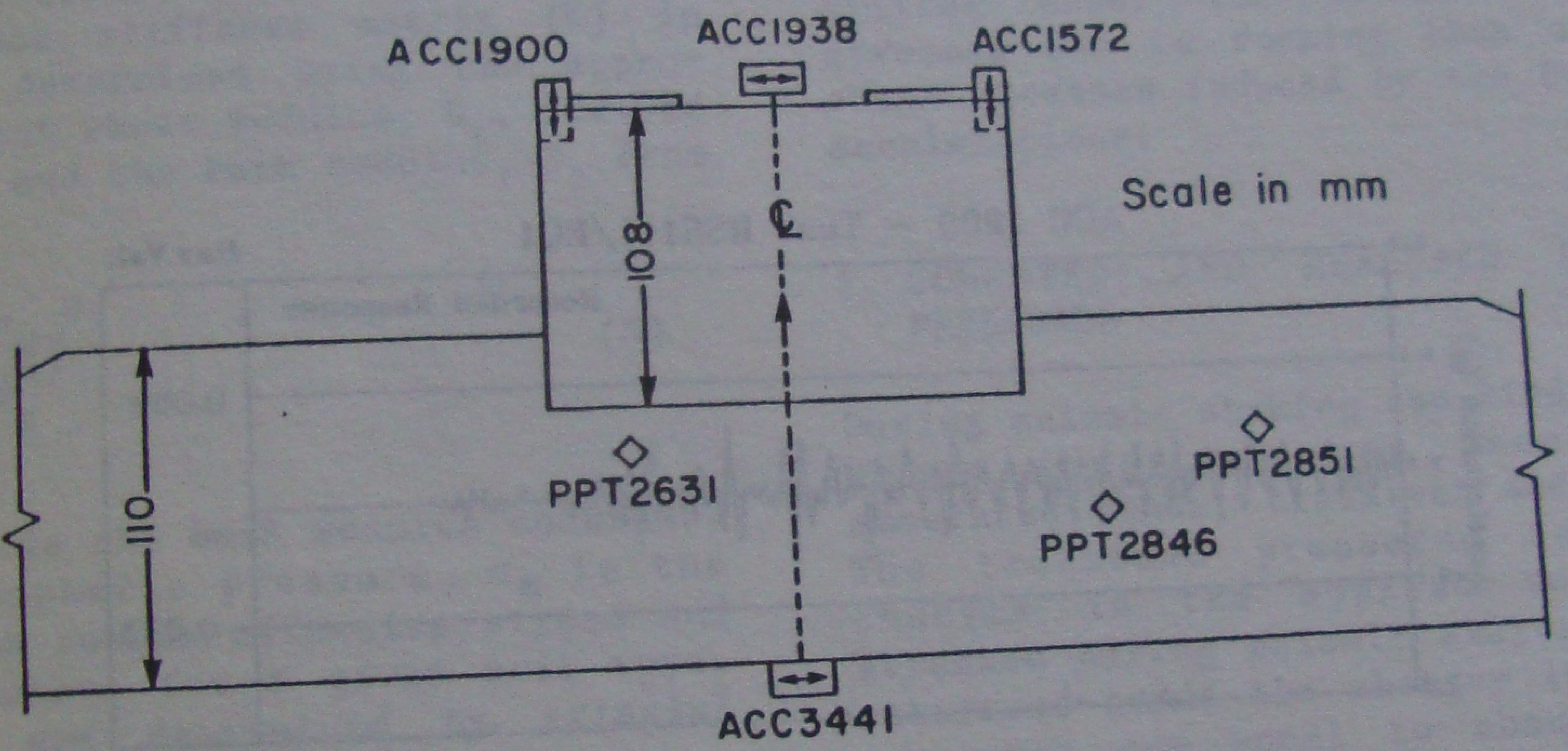


Figure 2. Locations of Transducers in Centrifuge Model.

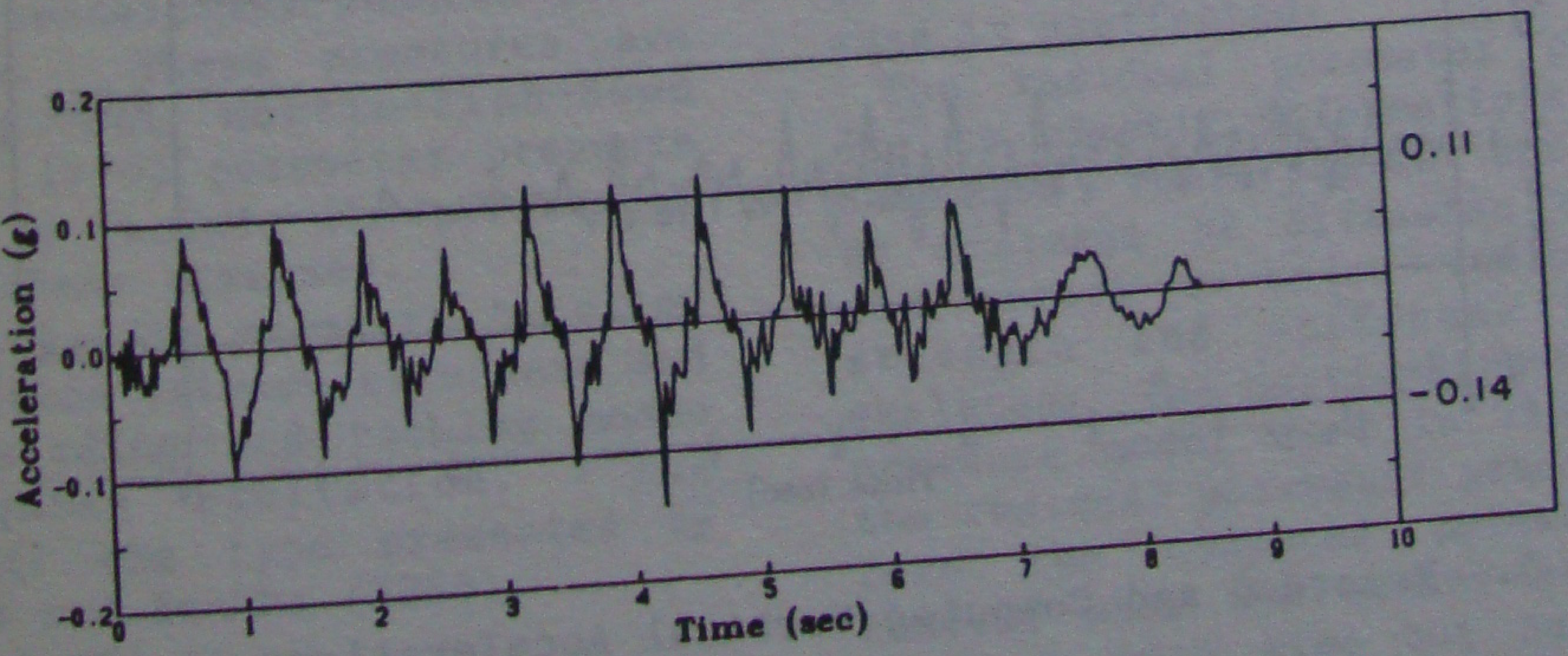


Figure 3. Input Motion Recorded by ACC 3441.

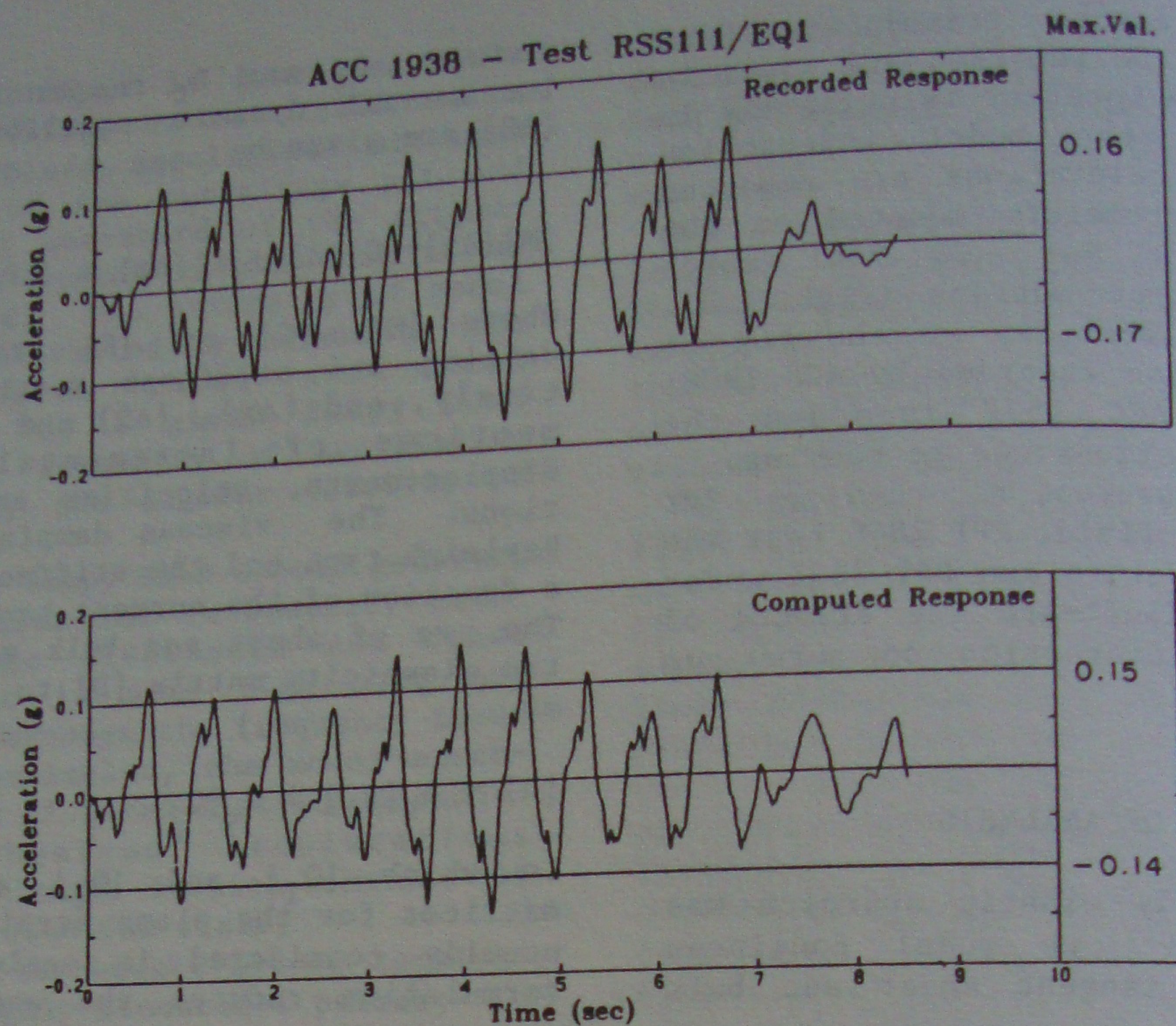


Figure 4. Recorded and Computed Horizontal Accelerations at ACC 1938.

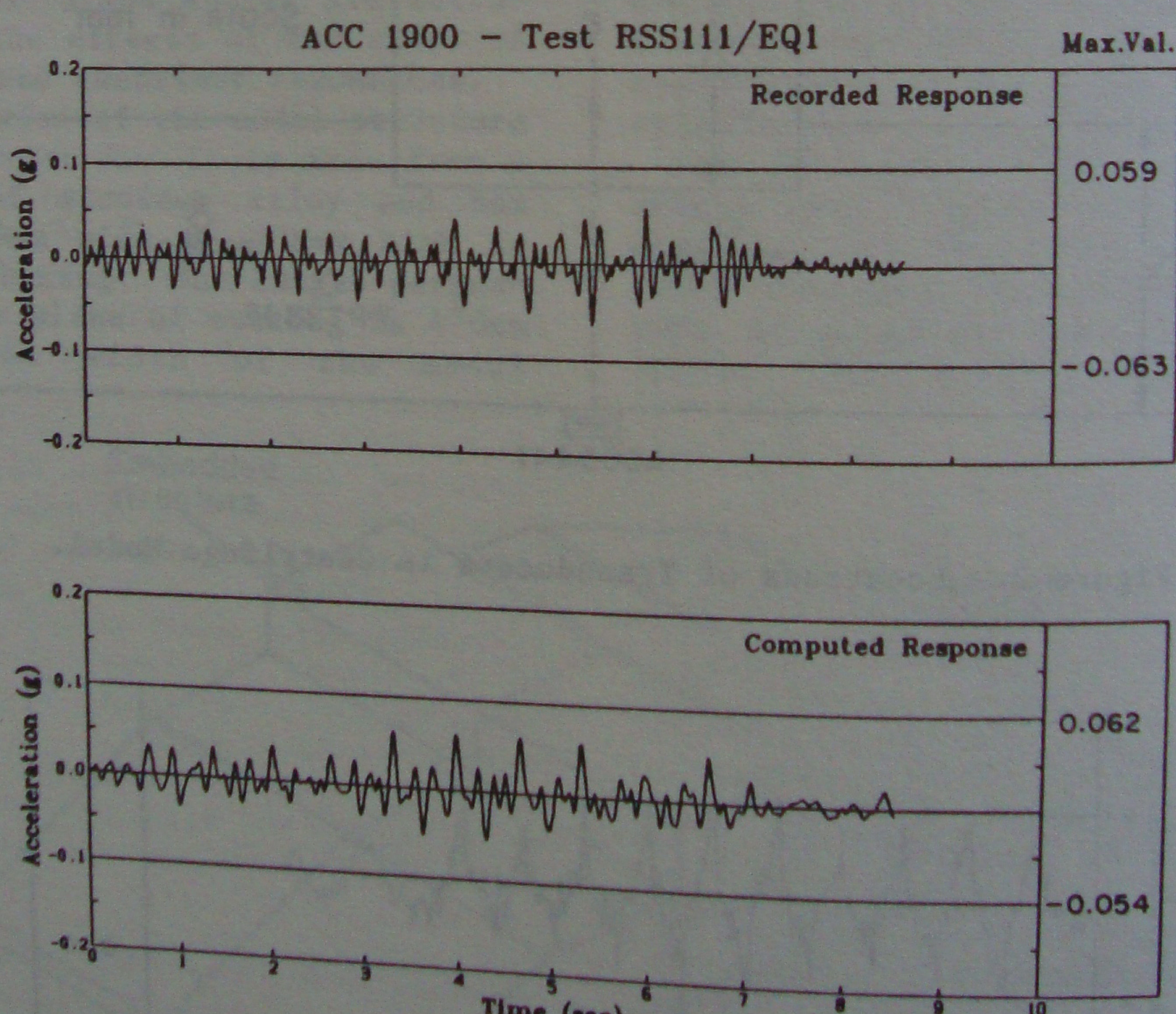


Figure 5. Recorded and Computed Vertical Accelerations at ACC 1900.

for computing [D] whenever $G_t$ and $B_t$ change in magnitude because of non-linearity or porewater pressure changes.

Behaviour in shear is assumed to be nonlinear and hysteretic. The response of the soil to uniform all round pressure is assumed to be nonlinearly elastic and dependent on the mean normal stress. In this deformation mode, hysteresis is neglected.

The relationship between shear stress, $\tau$, and shear strain $\gamma$, under either drained or undrained loading conditions is assumed to be hyperbolic and given by

$$\left(\frac{\tau-\tau_r}{2}\right) = \frac{G_{max}\,(\gamma-\gamma_r)/2}{1+(G_{max}/2\ \tau_{max})\,|\gamma-\gamma_r|} \qquad (3)$$

in which $G_{max}$ is the maximum shear modulus, $\tau_{max}$ is the appropriate shear strength and $(\gamma_r, \tau_r)$ is the point at which the loading last changed direction. This model of nonlinear hysteretic response assumes that unloading and reloading branches of hysteresis loops are defined from a skeleton curve using the Masing criterion (Masing, 1926).

The current stiffness matrix [K] in Eqn. 1 is determined using the appropriate tangent shear modulus, $G_t$, derived from Eqn. 3 and the bulk modulus, $B_t$ from

$$B_t = K_b P_a \left(\frac{\sigma_m}{P_a}\right)^n \qquad (4)$$

in which $K_b$ is the bulk modulus constant, $P_a$ is atmospheric pressure, $\sigma_m$ is the current mean normal effective stress and n is a constant for a given soil type. $K_b$ and n are determined by triaxial tests (Duncan and Chang, 1970).

Both $G_t$ and $B_t$ depend on the current mean-normal effective stress and must therefore be continuously modified for the effects of seismically induced porewater pressures. These pressures are computed using the Martin-Finn-Seed (Martin et al., 1975) porewater pressure model modified to include the effects of initial static shear stresses.

TARA-3 contains slip elements to allow for relative motion between structure and soil in both sliding and rocking modes during earthquake excitation. The elements are of the type presented by Goodman, Taylor and Brekke (1968).

## 4. COMPUTED AND MEASURED ACCELERATION RESPONSES

The soil-structure interaction model was converted to prototype scale before analysis using TARA-3 and all data are quoted at prototype scale. Soil properties were consistent with relative density.

The computed and measured horizontal accelerations at the top of the structure at the location of ACC 1938 are shown in Figure 4. They are very similar in frequency content, each corresponding to the frequency of the input motion given by ACC 3441. The peak accelerations agree fairly closely.

The vertical accelerations due to rocking as recorded by ACC 1900 and those computed by TARA-3 are shown in Figure 5. Again, the computed accelerations closely match the recorded accelerations in both peak values and frequency content. Note that the frequency content of the vertical accelerations is much higher than that of the horizontal acceleration at the same level in the structure and that of the input motion. This occurs because the foundation soils are much stiffer under the normal compressive stresses due to rocking than under the shear stresses induced by the horizontal accelerations.

## 5. COMPUTED AND MEASURED POREWATER PRESSURES

During seismic shaking two kinds of porewater pressures are generated in saturated sands; transient and residual. The transient pressures are due to changes in the applied mean normal stresses during seismic excitation. For saturated sands the changes in porewater pressures are equal to changes in the mean normal stresses. Since they balance each other, the effective stress regime in the sand remains largely unchanged and so the stability and deformability of the sand is unaffected.

The residual porewater pressures are due to plastic deformations in the sand skeleton. These persist until dissipated by drainage or diffusion and therefore they exert a major influence on the strength and stiffness of the sand skeleton. The Martin-Finn-Seed porewater pressure model used in TARA-3 generates the residual porewater pressures. Therefore the computed porewater pressure records show the steady accumulation of pressure with time but do not show the fluctuations in pressure caused by the transient changes in mean normal stresses.

The porewater pressures in the free field recorded by PPT 2851 are shown in Figure 6. In this case the changes in the mean normal stresses are not large and the fluctuations of the total porewater pressure about the residual value are relatively small. The dotted line shows the residual porewater pressures predicted by TARA-3. The peak residual porewater pressure, in the absence of drainage, is given directly by the pressure recorded after the earthquake excitation has ceased. In the present test, significant shaking ceased after 7 seconds. A fairly reliable estimate of the peak residual pressure is given by the record between 7 and 7.5 seconds. The recorded value is slightly higher than the value computed by TARA-3 but the overall agreement between measured and computed pressures is quite good.

As the structure is approached, the recorded porewater pressures show the increasing influence of soil-structure interaction. The pressures recorded by PPT 2846 adjacent to the structure (Figure 7) show larger oscillations than those recorded in the free field. This location is close enough to the structure to be affected by the cyclic normal stresses caused by rocking. The recorded peak value of the residual porewater pressure is given by the relatively flat portion of the record between 7 and 7.5 seconds. The computed and recorded values agree very closely.

The pronounced effect of large fluctuations in mean normal stresses on porewater pressure is exemplified by the record from PPT 2631 (Figure 8). This transducer was located directly under the structure and subjected to the large cycles of normal stress caused by rocking. There are relatively large shear strains adjacent to the base of the structure and the dilations resulting from these introduce additional fluctuations in the record. The porewater pressure recovery from the last major rocking cycle at 7 seconds is partly aborted by the post-earthquake drainage from around the structure so that the peak residual porewater pressure just after the earthquake is not accurately recorded in this case. It is given approximately by the top of the peak at about 7.5 seconds, close to the value computed by TARA-3.

## 6. CONCLUSIONS

This study demonstrates the utility of centrifuge modelling in providing a data base for validating methods of seismic response analyses. In no other way can such complete data coverage be obtained when required and at such a low cost. Key aspects of soil-structure interaction are clearly demonstrated in centrifuge tests such as high frequency rocking response, the effects of rocking on porewater pressure patterns and the distortion of free-field motions and porewater pressures by the presence of a structure.

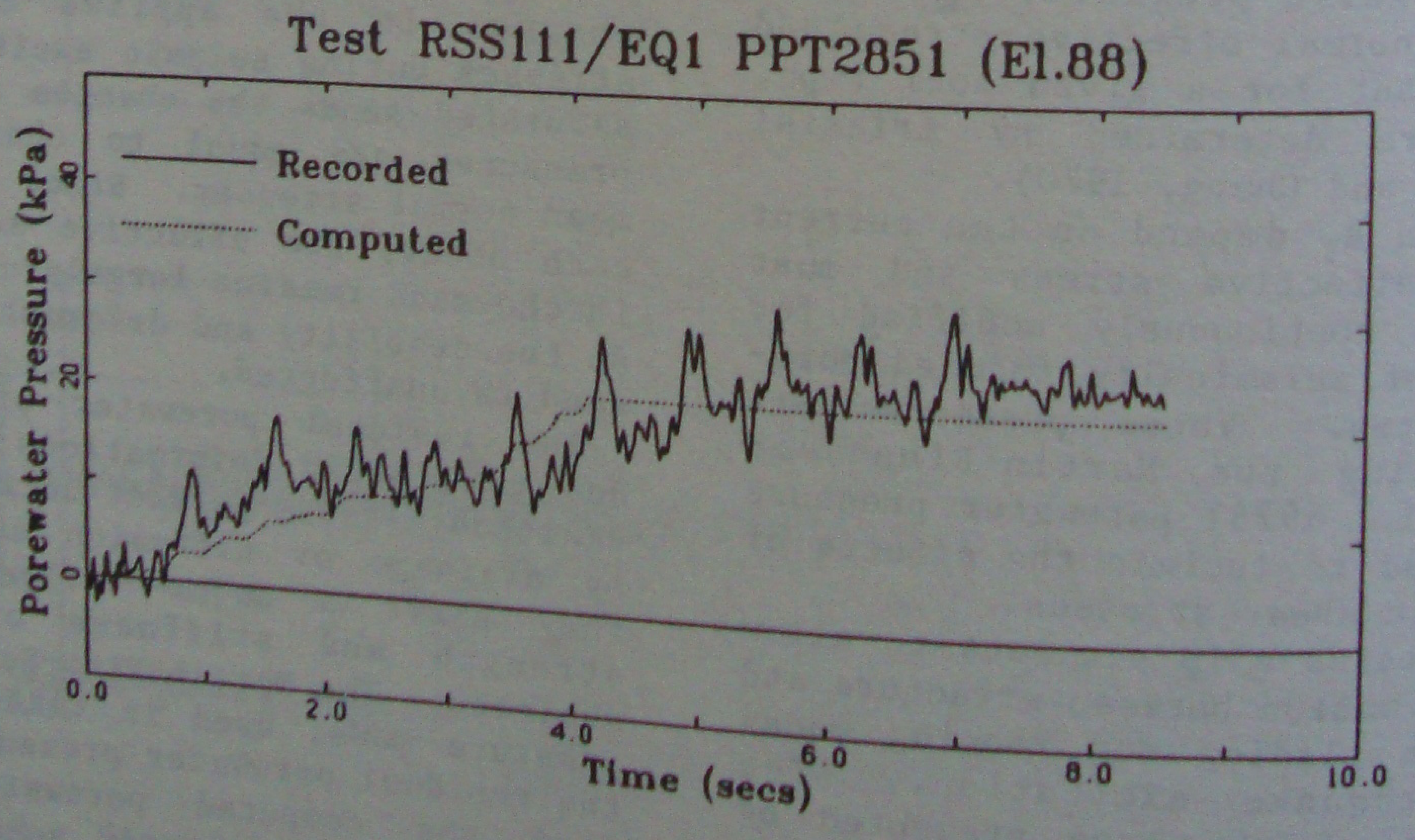


Figure 6. Recorded and Computed Powerwater Pressures at PPT 2851.

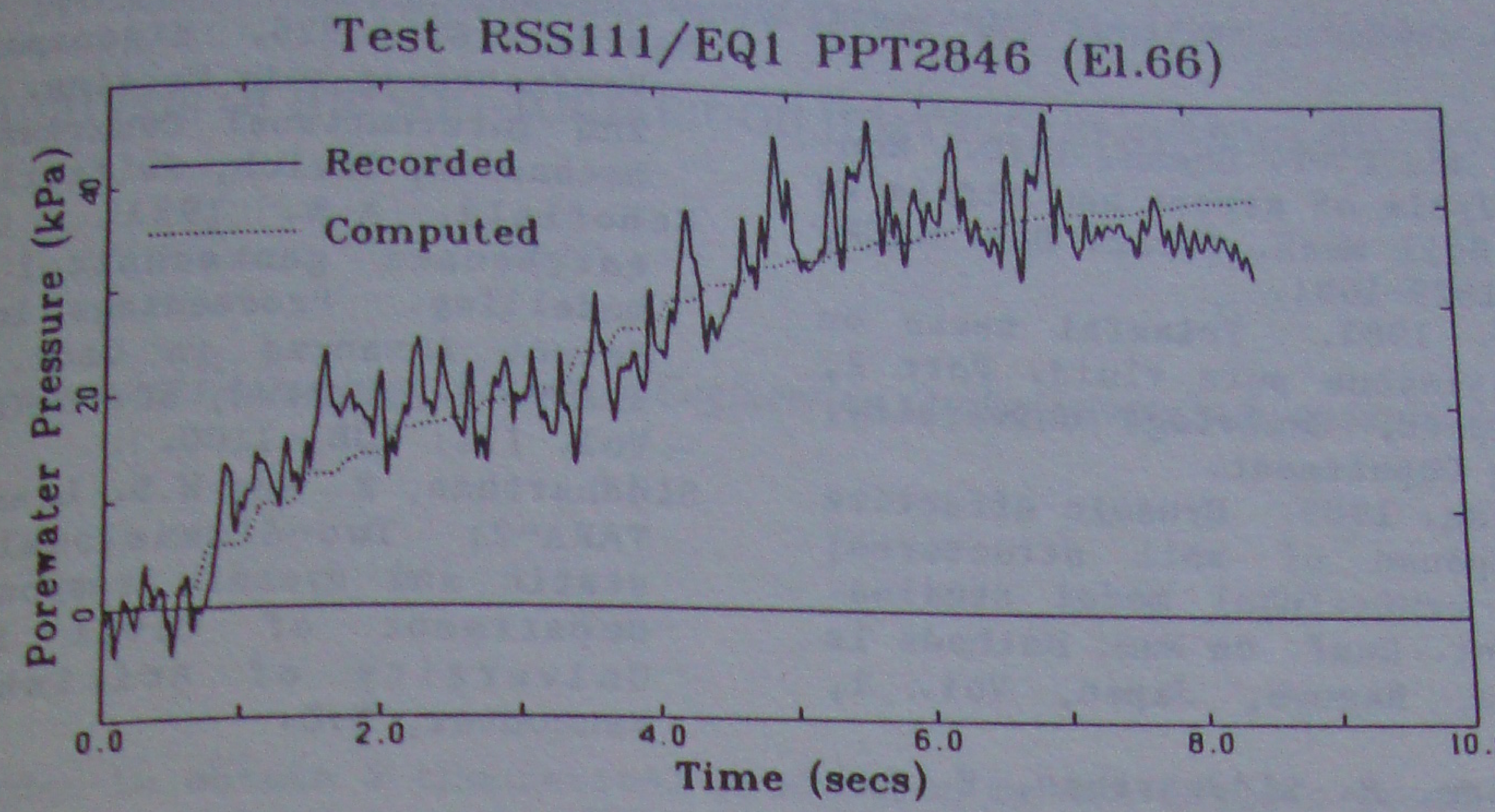


Figure 7. Recorded and Computed Porewater Pressures at PPT 2846.

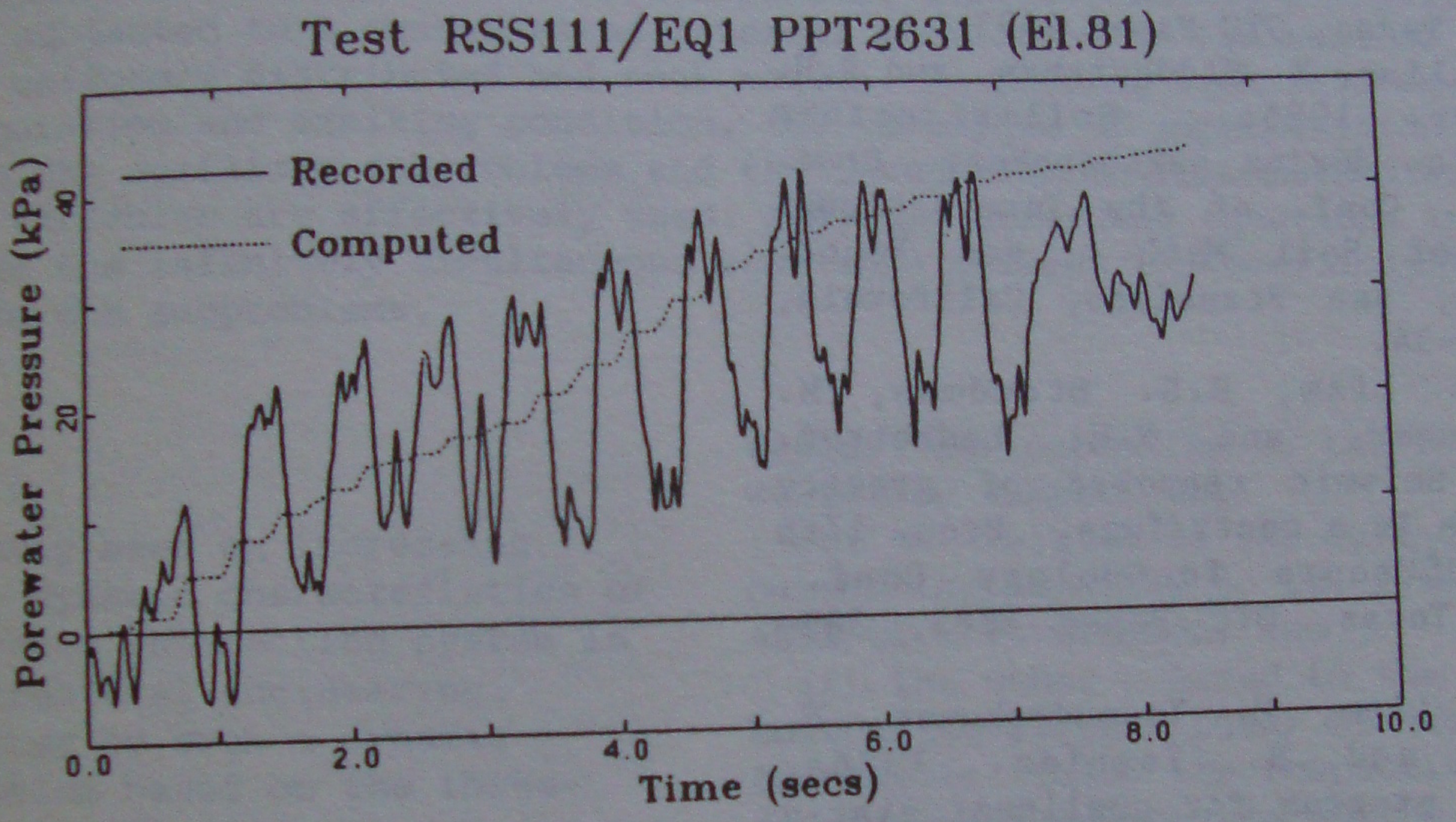


Figure 8. Recorded and Computed Porewater Pressures at PPT 2631.

The seismic response of a heavy structure embedded in a saturated sand foundation was satisfactorily modelled by the computer program TARA-3. The program incorporates a method for conducting nonlinear, hysteretic dynamic effective stress analysis. The accelerations in the foundation sands and in the structure computed by TARA-3 compared closely in magnitude and frequency content with recorded accelerations. In particular, the program was able to model the high frequency rocking vibrations. The residual porewater pressures in the foundation, both in the free-field and adjacent and under the structure were also modelled satisfactorily.

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