CANTERBURY EARTHQUAKES: THE RESTHAVEN RECORDS AND SOIL AMPLIFICATION RESPONSE

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At the Resthaven station, located in the northern part of the Christchurch central business district, acceleration ground motions were recorded during the 2010 September, Darfield (M_w 7.1) mainshock and the 2011 Christchurch earthquakes of February (M_w 6.3) and June (M_w 6.0). The elastic response and Fourier amplitude spectra are presented as the motivation for this study: the apparent low frequency content of the recordings could be a consequence of soil amplification (or another geotechnical earthquake induced effect). In this paper, a parametric series of equivalent linear analyses is performed to clarify the impact of soil amplification on record characteristics. Three soil profiles are adopted from the literature and investigated: (i) the V_S profile under the station, presented by Wood et al 2011; (ii) a more detailed profile but at a distance 300 m from the station, provided by Cubrinovski et al 2012; and (iii) a profile constructed on the basis of a SPT test performed by Beca Carter Hollings & Ferner Ltd 2011. Total-stress equivalent-linear and effective-stress inelastic analyses are performed. The results in the form of ground-surface acceleration time-histories and response spectra are compared to the actual recordings. Conclusions are drawn on the possible role of soil amplification and liquefaction.

Keywords: Christchurch earthquakes, soil amplification, total stress analysis, effective stress analysis.

1. INTRODUCTION: RESTHAVEN GROUND MOTIONS DURING THE CANTERBURY EQS

During 2010-11 several major earthquakes (of magnitude M > 5.7) rattled the Canterbury region most of which epicentered near Christchurch. The Darfield M_w 7.1, 4 September 2010, earthquake was the first and largest event of all, followed by a series of magnitude $M_w \approx 6.0$ (±) earthquakes. Four of these earthquakes are included in this study:

- Darfield EQ: $M_W = 7.1$, 4 September 2010
- Christhurch EQ: $M_L = 6.3$, 22 February 2011

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- Off Christchurch EQ: M_W = 6.0, 13 June 2011
- Off Coast Christchurch EQ: M_L = 6.0, 23 December 2011

These earthquakes were generated on completely unknown and unsuspected faults, surprising an earthquake conscious nation. More than 15 accelerograph stations, well-distributed in the city and the surrounding communities, recorded the events, offering invaluable ground motions. Figure 1 illustrates a map with the locations of the recording stations at Christchurch city, focusing at the City Building District (CBD) area. The REHS records will be the theme of this paper.

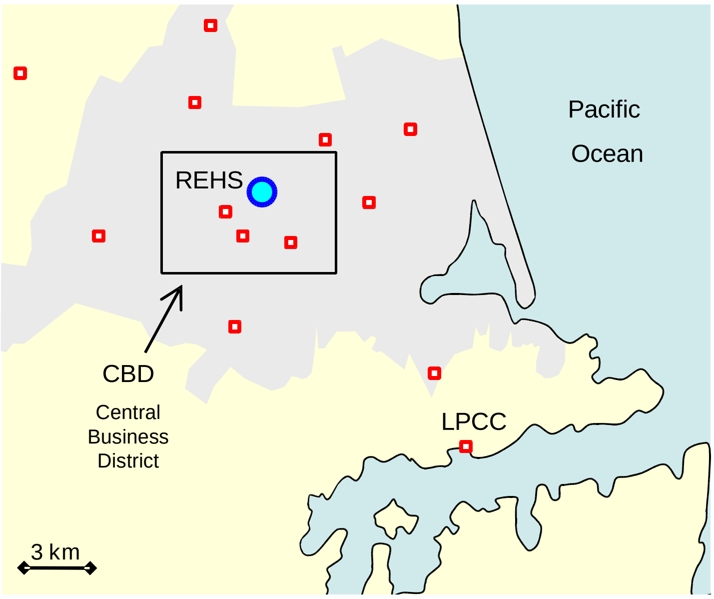

Figure 1. Map of Christchurch area with the location of the Resthaven station (solid circle) and the rest seismograph stations (squares).

The records utilized herein were taken from the NGS Strong Motion Data Base, through GeoNet (ftp://ftp.geonet.org.nz/strong/processed/Proc/2011/). The REHS seismograph is located in a separate storage within the Resthaven facilities. The recorded motion is considered a truly free-field response taking into account the open space around the station (Tasiopoulou et al., 2012).

Figure 2 depicts the north-south (NS) acceleration components of the record, together with the elastic (5% damping) acceleration, S_A, and velocity, S_V, response spectra. Furthermore, the Fourier amplitude, $\mathcal{A}$, spectrum is presented. From the acceleration time-histories it can be observed: (a) the small variation of peak ground acceleration: (0.25 ÷ 0.35) g, and (b) the different duration of the four records in function of the event magnitude. Particularly for the February event, acceleration decrease and period elongation after t = 10 s can be noticed in the record, hinting at soil softening, perhaps due to liquefaction. Only a limited amount of sand boiling had emerged on the ground surface following the earthquake, according to eyewitness reports. The remaining records in CBD are of similar intensity and nature, with more-or-less the same manifestation of amplification/liquefaction.

The damped elastic response spectra, S_A and S_V, offer a complete visual assessment of the potential of a ground motion to cause large response to (visco)elastic spring–mass systems. Figure 2 shows that the main peak of the acceleration spectra occurs at about 1 s to 1.5 s, which is indicative of soft soil conditions. What is more, the velocity spectra present a period-shift of maxima to even longer periods (1.5 to 2 s). The rich long period content of REHS records is further portrayed in Fourier amplitude spectra: the first region of large Fourier amplitudes can be noticed for the period range of 1÷2 s, while the second maxima region presents for even larger periods of 2.5÷4 s.

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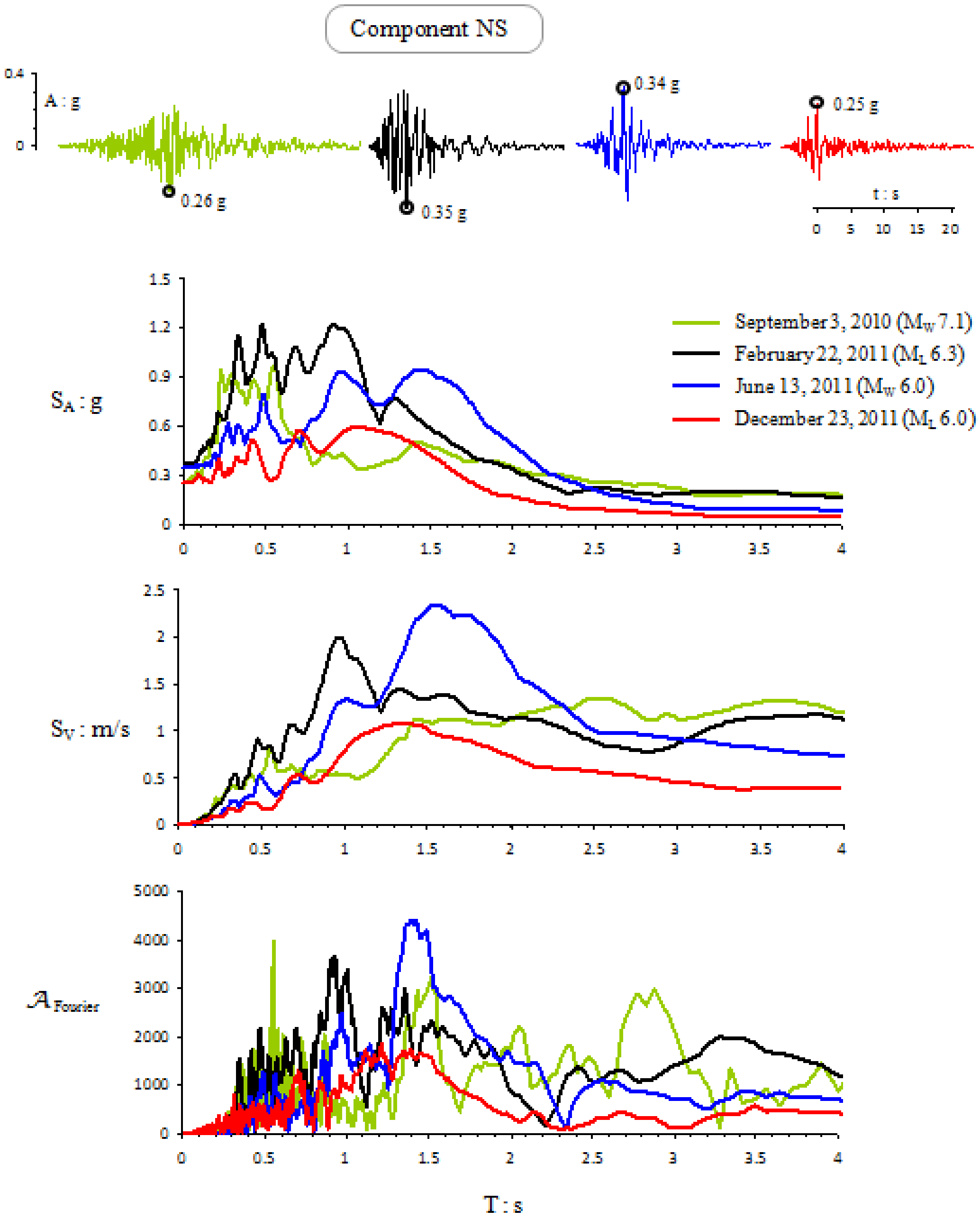

Figure 2. The Resthaven recorded ground motions in the Canterbury Eqs. Acceleration time histories on the top, plotted at the same scale. Elastic acceleration [S_A], velocity [S_V], and Fourier amplitude [$\mathcal{A}$] spectra.

2. REHS SOIL PROFILES AVAILABLE FROM THE LITERATURE

Christchurch sits on the eastern edge of the 80 km wide, fluvial, Quaternary Canterbury Plains. The central city and eastern suburbs are built on this layer, and although the former swamps and lagoons are now drained, the water table remains shallow.

The soil in REHS consists of more-or-less granular soil layers. The water table is one meter below the surface. At present, three soil profiles of the Resthaven site are available from the literature.

First, Wood et al. (2011) performed active and passive-source surface wave testing to obtain shear wave velocity, V_S, profiles at 13 strong motion station sites in the greater Christchurch area. Figure 3(a) portrays the V_S soil profile of Resthaven station.

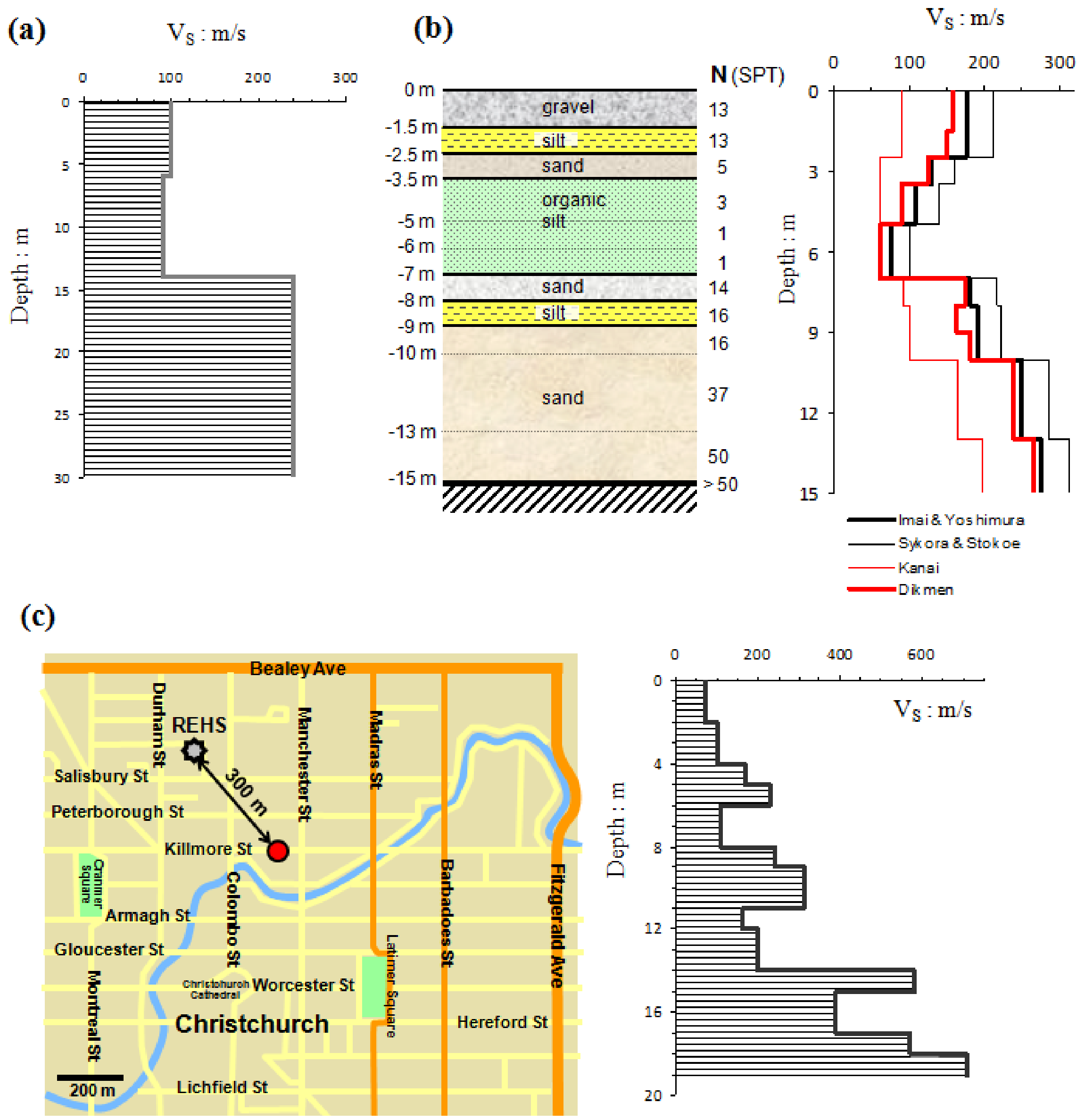

Figure 3. Shear wave velocity profiles utilized in our study: (a) by Wood et al. (2011), (b) the SPT borehole (BECA 2011) and corresponding profiles using empirical correlations, and (c) by the work of Cubrinovski et al. (2012).

Second, Beca Carter Hollings & Ferner Ltd (Beca) undertook a soil investigation for the New Zealand Department of Building and Housing. One of its sites was outside the REHS station building. The investigation comprised standard penetration testing to 15 m and cone penetration testing to 20 m. Figure 3(b) describes the profile of standard penetration resistance N-values with depth. We utilise the N values and several empirical shear wave velocity correlations to obtain shear wave velocity profiles.

Third, Cubrinovski et al. (2012) provide the V_S soil profile shown in Figure 3(c), as a result of CPT tests performed at a site 300 m south-east of the REHS recording station.

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3. BEDROCK EXCITATION AT RESTHAVEN SITE

To define the basement motion to be used as excitation the LPCC record, from a station in the port of Lyttelton placed next to volcanic rock outcrop, was considered. Utilizing the soil V_S profile of Wood et al. (2011), which revealed the presence of 6 m of stiff soil above the rock, the LPCC ground motion was deconvoluted to bedrock level, as illustrated in Figure 4. The surface accelerogram of PGA = 0.78 g, was reduced to the rock outcrop accelerogram of PGA = 0.59 g. The latter will be used from now on as the base excitation of all total- and effective-stress analyses performed. The acceleration response spectra of the deconvoluted bedrock motion together with the recorded LPCC motion are depicted in Figure 5.

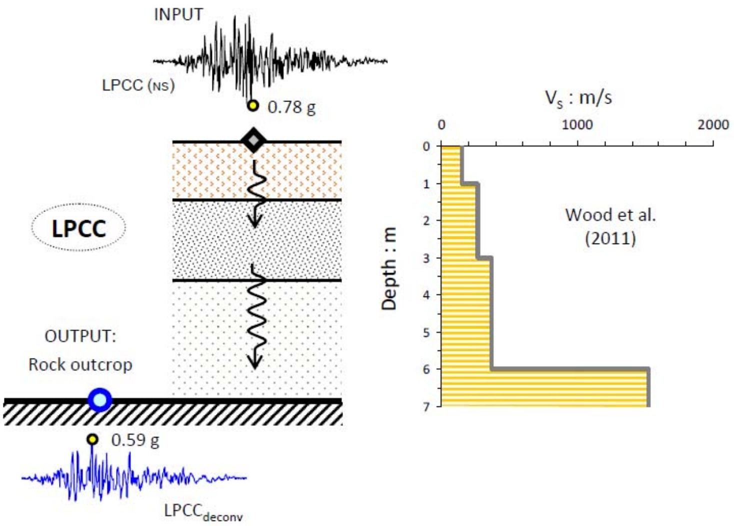

Figure 4. Soil profile of the LPCC station (by Wood et al. 2011) and deconvolution analysis to maintain the bedrock motion, which will be utilized as excitation at the base of the Resthaven profile.

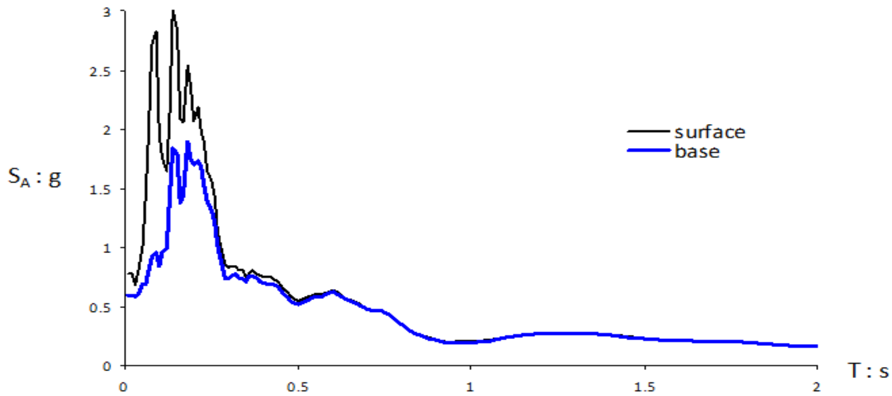

Figure 5. Acceleration response spectra of the recorded ground motion at the surface of LPCC station (black solid line) and of the deconvoluted bedrock excitation (blue solid line).

4. TOTAL–STRESS AND EFFECTIVE–STRESS ANALYSES: RESULTS AND COMPARISONS

To evaluate the effect of local soil conditions on REHS ground response, equivalent-linear SHAKE wave propagation analyses are first performed. Nonlinear soil behaviour is incorporated by applying hysteretic G-γ and ξ-γ curves. Several material curves are utilised to describe with detail the dynamic

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characteristics of sand, gravel, and silt [Vucetic & Dobry (1991), Seed & Idriss (1970), Gazetas & Dakoulas (1992), Sun et al. (1988) and Ishibashi & Zhang (1993)]. The soil profile was enhanced by a 400 m stiff gravelly layer. Summarised results are presented in graphical form in Figure 6(b). It is evident that the high spectral values of the record around T ≈ 1 s and T ≈ 1.3 s can not be explained by 1-D soil amplification of total-stress analysis.

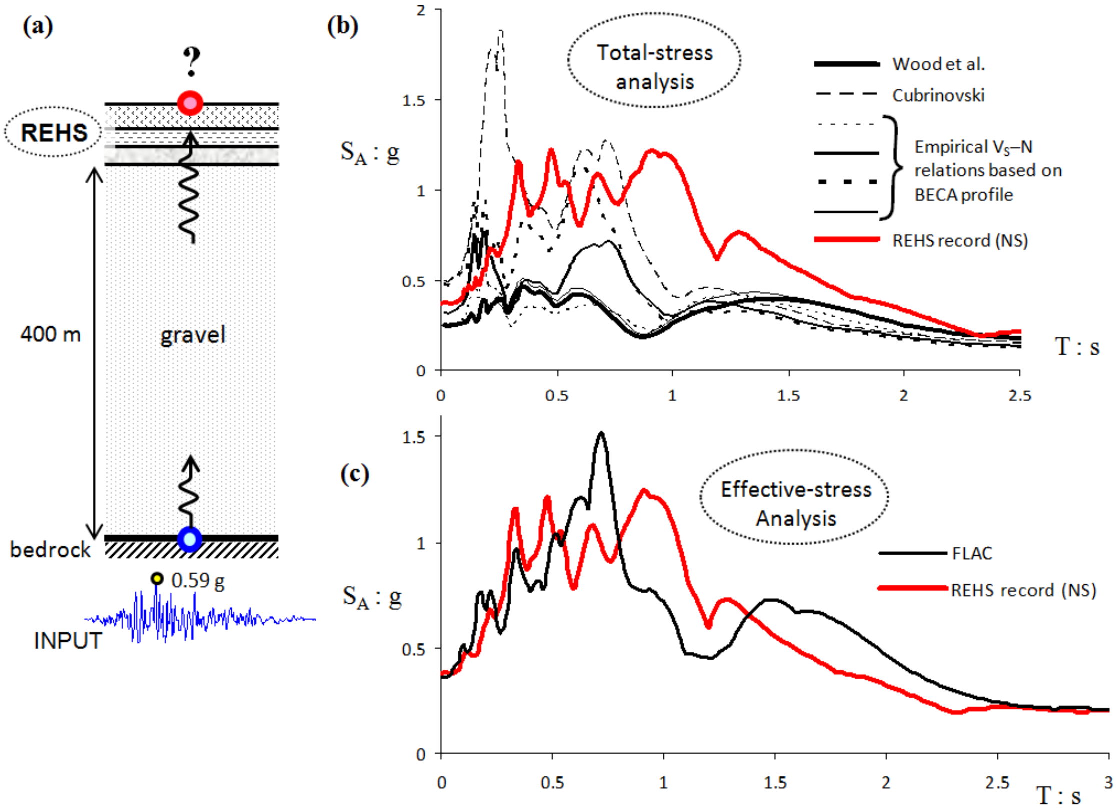

Figure 6. Parametric analysis of soil amplified response for REHS site: (a) "generic" soil profile and excitation, (b) soil surface response spectra from total-stress analyses versus the recorded motion response, (c) soil surface spectra from effective stress analysis comparing to the recorded motion.

To investigate the soil response accounting for pore-water-pressure development and perhaps liquefaction, the soil profile of Figure 3(b) was chosen. Dynamic effective-stress analyses were conducted in order to capture the excess pore water pressure rise and dissipation, using the finite difference code FLAC (Itasca, 2005). The numerical simulation involves the UBC sand constitutive model (Beaty and Byrne, 1998), assigned to the sand/silt layers to allow for pore pressure generation. Calibration of the model was based on the SPT values.

The surficial soil layers play a dominant role in defining the ground motion characteristics: these layers behave as a filter cutting-off the high frequency spikes, while the period of motion cycles is lengthened. In terms of spectral acceleration values, there is considerable amplification in the higher period range of 1.5 s to 2 s [Figure 6 (c)].

The occurrence of liquefaction is clearly visible in the pattern of both numerically-obtained and, especially, recorded ground acceleration histories after 8 s [Figure 7(b)]. At that point onward the soil loses most of its strength and filters out the high frequency components, cutting off the acceleration values, and allowing only (long–period) oscillations of the dry cover layer that is "floating" on the top of the liquefied layer. At that time, the ratio r_u of the earthquake-generated (excess) pore water pressure, Δu, normalized by the initial vertical effective overburden stress, σ'_{vo},

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$$r_u = \frac{\Delta u}{\sigma'_{vo}} \qquad (1)$$

approaches value of 1, as illustrated in Figure 7(a). It is worth mentioning that liquefaction is reached at a later time, just before the end of strong shaking. In addition, the two time histories depicted in Figure 7(b) have many common features: similar PGA values, frequency content, and especially they “share” a significant 1.5 second duration pulse starting at 4 s. In terms of acceleration spectra [Figure 6(c)], these similarities lead to amplification at periods about 1.5 s, as also shown by Smyrou et al. (2012), while the “late” liquefaction allowed for amplification at shorter periods.

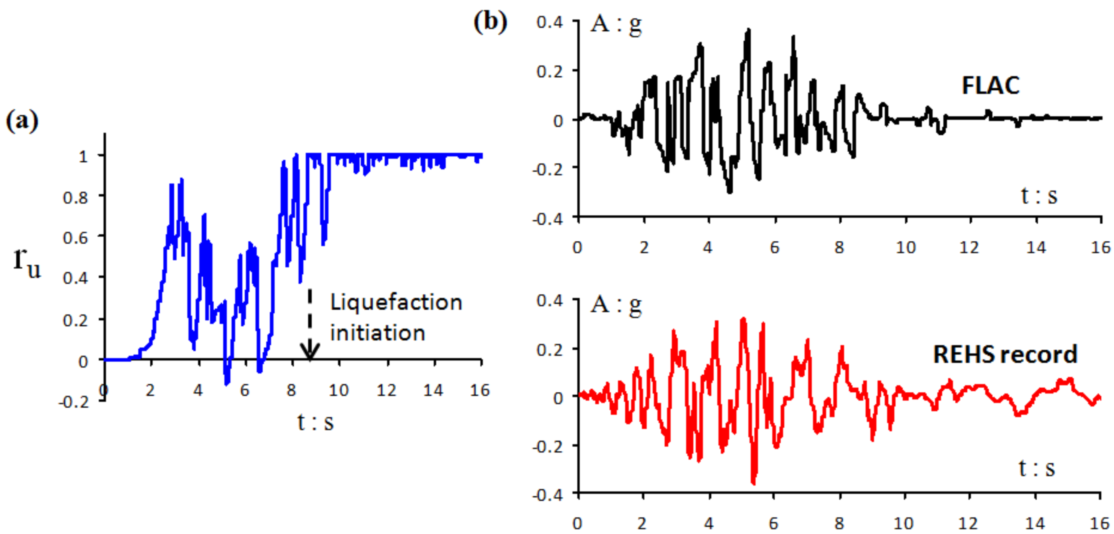

Figure 7. (a) Time history of excess pore pressure ratio 3m below the ground surface (in the middle of the upper sand layer), (b) comparison between acceleration time history obtained by numerical analysis (FLAC) on the soil surface and the recorded motion.

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