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NON LINEAR SITE RESPONSE SIMULATION DURING THE 1999 CHI-CHI EARTHQUAKE IN TAIWAN

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SUMMARY

This paper presents an application of the non-linear multi-kinematics dynamic model implemented in *CyberQuake* software (Modaressi [1]), analysing the seismic response at a site located within the Chang-Hwa Coastal Industrial Park during the 1999 Chi-Chi earthquake in Taiwan.
Computed NS, EW and UP ground accelerations obtained with this model under undrained and two-phase assumptions, are in good agreement with the corresponding accelerations recorded at seismic station TCU117, either for peak location, amplitudes or frequency content. During simulation, liquefaction occurs between depths 1.3m and 11.3m, which corresponds to the observed range attested by in place penetration tests and liquefaction analyses.

INTRODUCTION

Observations from many recent strong motion events, such as the 1989 Loma Prieta, the 1994 Northridge and the 1995 Kobe earthquakes, have shown the importance of local geological site conditions on the seismic ground response. These events have also demonstrated that nonlinear soil behaviour strongly affects the seismic motion of near-surface deposits, resulting in shear wave velocity reduction, irreversible settlements and in some cases, pore-pressure build-up leading to liquefaction.

Since the early 70s', most studies of moderate and strong motion assessment concerning soft soils are performed using the equivalent linear approach (Idriss [2], Seed [3]) based on a visco-elastic multi-layered soil model. But it is now well-admitted that this model is not adapted when shear strain amplitude is exceeding 10^{-3}. An appropriate non-linear constitutive model for soil deposits, such as the strain-hardening cyclic elastoplastic constitutive model implemented in *CyberQuake* software (Modaressi [1], Mellal [4]), is thus recommended, to be able to reproduce the complex features of soil behaviour under seismic loading (Foerster [5]). The non-linear multi-kinematics dynamic model implemented in *CyberQuake*, well-suited for multi-layered soil profiles under various hydraulic conditions (totally drained, partly drained or fully undrained conditions), is able to reproduce shear wave velocity reduction, irrecoverable settlements and pore-fluid pressure build-up leading to liquefaction.

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In this paper, the seismic soil response of a site located within the Chang-Hwa Coastal Industrial Park, was simulated during the 09/21/1999 Chi-Chi earthquake in Taiwan (M_w = 7.5). Large settlements (33-45cm), as well as evidence of liquefaction were observed at this site. Liquefaction was attested in the 4-9m depth interval by penetration tests (up to 14m in some places), sand boils and unusual wet ground surface (Lee [6]). Through this case study, we demonstrate the importance of using appropriate constitutive modelling when the part played by non-linear phenomena is preponderant in the site effect analysis. For instance, as in place geotechnical data were available before and after the seismic event (standard and cone penetration tests), it was possible to calibrate the constitutive model parameters and then to confront the results from simulation to the observations, essentially the shear modules along the soil profile and the ground accelerations and displacements.

THE NONLINEAR DYNAMIC MODEL

The transient analysis

The soil profile is modelled as a multi-layered drained or saturated deformable porous medium with homogeneous laterally semi-infinite layers lying over a rigid or elastic bedrock. This one-dimensional geometry remains valid when no lateral heterogeneity, either geometric or material, does exist. The model kinematics is three-dimensional, with two horizontal and one vertical components of motion.

The transient non-linear dynamic model implemented in *CyberQuake* software is based on the well-known simplified *u-p* formulation and uses Terzaghi's effective stress principle (Modaressi [1]). When a water table is present in the soil profile, layers located above the water level are assumed to be totally drained, whereas a fully undrained or a partially drained (two-phase) condition can be considered for the saturated layers underneath. In the coupled two-phase approach, both phases (water and solid) are assumed to be incompressible. This approach is useful when conducting a long term analysis (for instance, post-seismic consolidation) and/or when high contrasts in permeability values are encountered in the soil profile.

Concerning numerical aspects, the governing equations are discretized by finite elements and integrated with respect to time by an explicit predictor-corrector Newmark scheme. An adaptive time step is computed automatically by the software, in order to fulfil stability and accuracy requirements for the integration scheme.

The constitutive model

The cyclic elastoplastic constitutive relationship implemented in *CyberQuake* is successful in reproducing the complex features of non-linear soil behaviour under seismic loading. This model (Modaressi [1], [7], Mellal [4]), is based on the original model developed by Aubry [8]. Its main characteristics are as following:

- a unique *Coulomb*-type failure criterion assumed for both monotonous and cyclic loading paths;
- a progressive mobilisation towards plasticity: the plastic yield surface evolves within a kinematic strain-hardening regime, which depends on the consolidation pressure (as in the *Cam-Clay* family models) and on the plastic distortion;
- a realistic dilatant/contractant soil behaviour, through a *Roscoe*-type dilatancy rule defining the evolution of the plastic strain rate.

A post-seismic consolidation phase

CyberQuake is also able to perform a post-seismic consolidation analysis, using a two-phase coupled approach. Simulation in this phase resumes from the end of the previous seismic computation and is conducted until the pore-fluid over-pressures due to the shaking, are totally dissipated in the soil profile. Resulting additional settlements and deformations are also determined in this phase.

SOIL PROFILE CHARACTERISTICS

Geotechnical characteristics of the site

The chosen site is located in the West 2nd District of Lukung (west coast of central Taïwan), which is in the Chang-Hwa Coastal Industrial Park. A campaign of standard penetration tests (SPT) and cone penetration tests (CPT) was conducted within the top 30m of the site before the 1999 earthquake event. The groundwater table is generally within the top 2m, and was located at 1.12 m depth in our study (Lee [6]). After the earthquake, additional SPTs and CPTs field experiences were conducted on the site (same location as previous tests), and especially in the vicinity of observed sand boils.

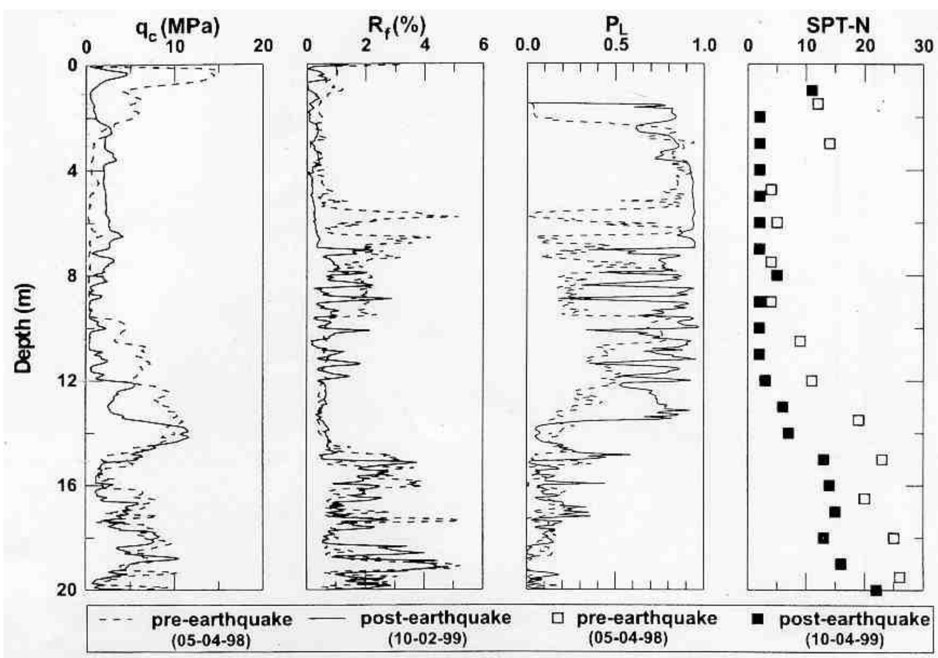

Fig. 1: Results of CPTs and SPTs in the liquefied area before and after the 21 September 1999 Chi-Chi earthquake (after Lee [6]).

SPTs and CPTs field data were used to estimate the maximum shear modulus (G_{max} in kPa), by considering the existing correlations for sands and clays:

- SPT: $G_{max} = 357\ p_a \left(N \frac{\sigma_v'}{p_a} \right)^{1/3}$ for sands (after Seed [2]) and $G_{max} = 357\ p_a \left(N \frac{\sigma_v'}{p_a} \right)^{1/3} \cdot OCR^k$ for clays (after Hardin [9]),

with σ_v', the effective vertical stress in kPa; p_a, the atmospheric pressure (= 100 kPa); N, the standard penetration resistance varying between 5 and 50; and OCR, the over-consolidation ratio and k, an exponent varying with the plasticity index of the clay.

- CPT: $G_{max} = 1634\ q_c \left(\frac{q_c}{\sqrt{\sigma_v'}} \right)^{-0.75}$ for sands (after Lunne [10]), with q_c, the tip resistance in kPa.

The shear wave velocities V_s were calculated within the soil layers from $G_{max} = \rho V_s^2$, with ρ, the estimated bulk density and P-wave velocities V_p were derived from V_s with: $V_p = V_s \sqrt{\frac{2(1-\nu)}{1-2\nu}}$, where ν is the estimated Poisson ratio (see Table 1).

Table 1: Studied soil profile (groundwater table at 1.12m depth).

Layer	Description	Depth (m)	V_S (m/s)	V_P (m/s)	ρ (kg/m^3)	Hydraulic Conductivity (m/s)
1	Backfill (gravel)	1.12	135	254	2000	-
2	Sand or silty sand	1.5	150	281	2000	10^{-5}
3	Sand or silty sand	3	153	287	2200	10^{-5}
4	Sand or silty sand	4.5	140	262	2200	10^{-5}
5	Sand or silty sand	7.5	144	269	2200	10^{-5}
6	Sand or silty sand	9	145	271	2200	10^{-5}
7	Sand or silty sand	10.5	178	334	2200	10^{-5}
8	Sand or silty sand	12	205	381	2200	10^{-5}
9	Sand or silty sand	15	221	414	2200	10^{-5}
10	Clay or silt with sand	18	252	471	1900	10^{-7}
11	Clay or silt with sand	21	262	498	1900	10^{-7}
12	Clay or silt with sand	24	278	520	1900	10^{-7}
13	Clay or silt with sand	27	239	447	2000	10^{-7}
14	Clay or silt with sand	28.5	248	464	2000	10^{-7}
15	Sand or silty sand	30	281	526	2000	10^{-5}
Bedrock	Elastic Bedrock	-	300	550	2200	impervious

Input motion

In this analysis, the three components of motion recorded at the nearby seismic station TCU109, considered here as an outcropping bedrock, were used as the input motion for the studied site. According to the site classification of Taiwan free-field strong-motion stations (Lee [11]), this station is indeed on stiff soil of class D. The observed peak ground acceleration (PGA) was equal to 0.15g on this site. Moreover, a maximum frequency of 6Hz was assumed for the analysis.

Ground accelerations computed for the studied site were compared to the accelerations recorded at the Shanshi Middle School (station TCU117), a station located on soft soil (class E according to the aforementioned classification), about 3 km away from the site and on which the PGA is about 0.12g (Juang [12]).

Non-linear model parameters

Calibration of the constitutive model parameters was performed using a specific tool (the "Parameter Wizard") implemented in *CyberQuake*. This tool requires a few standard geotechnical data as input and produces the parameters using various correlations from literature, depending on the material type considered. For instance, determination for sands and gravels is mainly based on the granulometric features (granulometric range and grain shape) and relative density, whereas determination for clays is based on the plasticity index and OCR.

Additional reported qualitative information and liquefaction analysis performed on the site (Lee [6]), also helped to determine non linear characteristics within the soil profile. The analysis notably highlighted high liquefaction potential within the top 13m, and particularly between 2.5 and 6m. Moreover, the analysis of the CPTs conducted on the site after the earthquake, shows that little change in q_c (value and pattern) are observed below 14m depth, indicating that liquefaction occurred within the top 14m at this location. Finally, the observed ground settlement was in the range of 33-45cm.

NUMERICAL RESULTS AND DISCUSSION

Comparison between the three components of acceleration computed at ground surface with the undrained and two-phase non linear approach, the undrained elastic approach and recorded at station TCU117, are shown on Figure 2. The corresponding response spectra are shown on Figure 3. Peak locations are in good agreement, especially the two-phase simulation reproduces closely the response spectra of the recorded accelerations. A slight amplification of the amplitudes occurs at ground surface (computed PGA of 0.15g instead of 0.12g for the recorded data). The difference may be attributed to our insufficient knowledge of the geomechanical properties for the layers underlying station TCU117. In fact, although this station is the nearest station to the studied site, it is located a little far from it (3 km) and underlying properties may vary from the soil profile considered in this analysis. At last, comparison between the elastic and non linear approaches demonstrates that non linear behaviour must be taken into account to obtain accurate results.

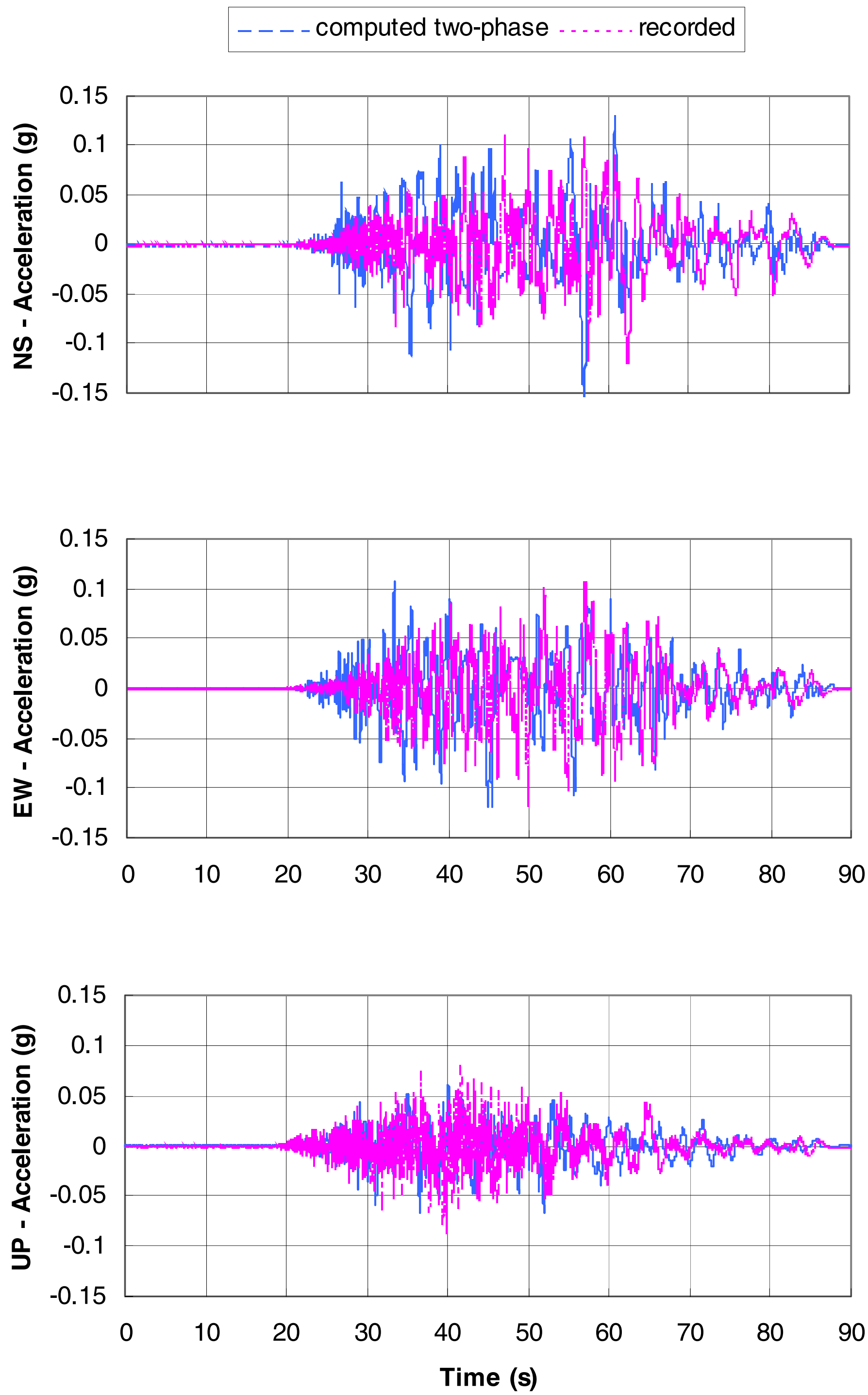

Fig. 2: Computed and recorded accelerations (station TCU117) at ground surface

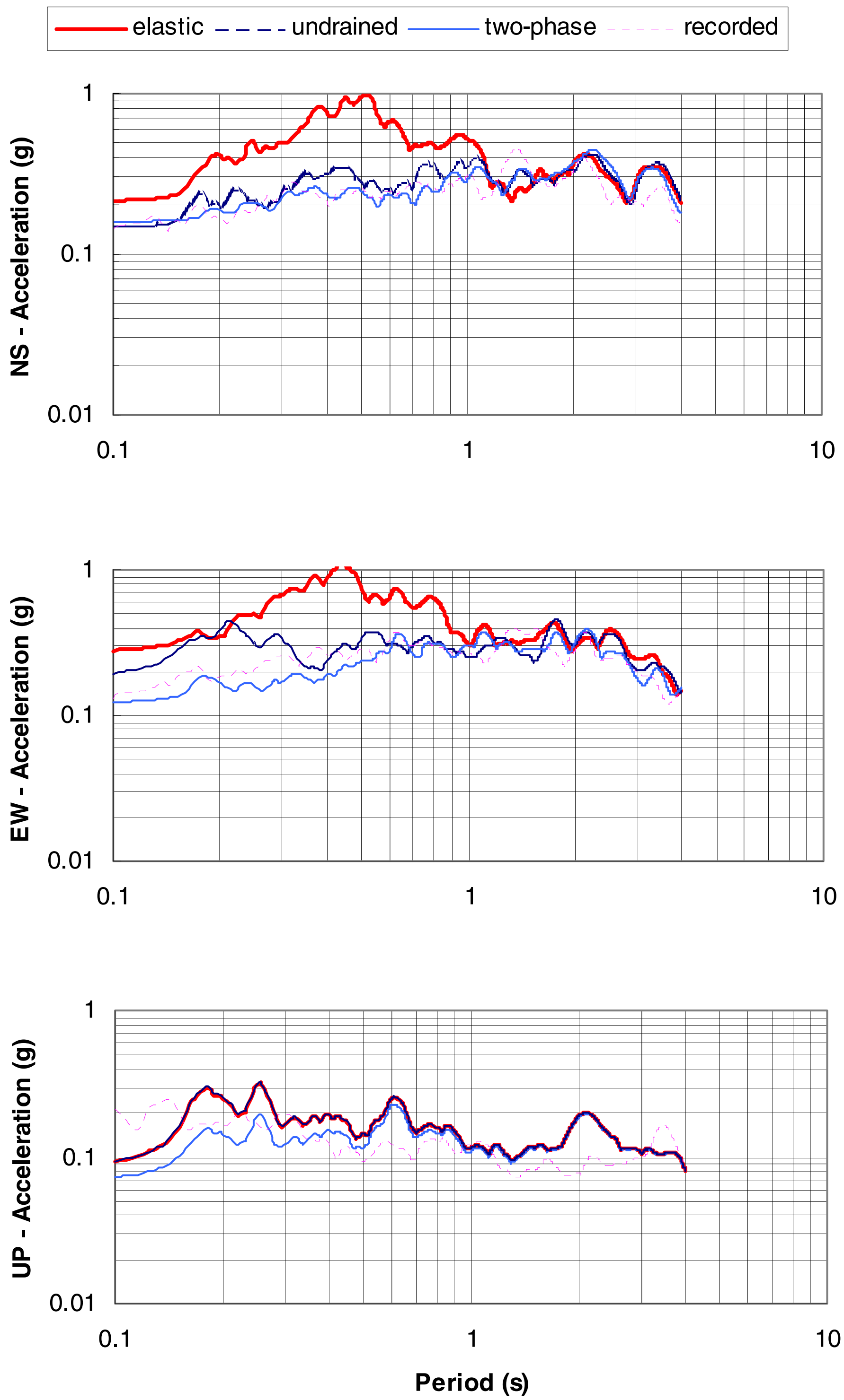

Fig. 3: Computed and recorded response spectra in acceleration at ground surface

Moreover, the two-phase non linear simulation leads to liquefaction between 1.31m and 13.5m, which is in good agreement with the field tests carried out after the earthquake. These tests highlighted high liquefaction potential within the top 14m, and particularly between 2.5 and 6m. Looking at pore-fluid over pressure ratio values (Figure 4), we see that liquefaction occurs after about 32 seconds of motion at 13.5m, and after 47 seconds at 7.5m depth. Liquefaction can also be observed on shear stress vs. normal effective stress diagrams on Figures 5 (2D) and 6 (3D).

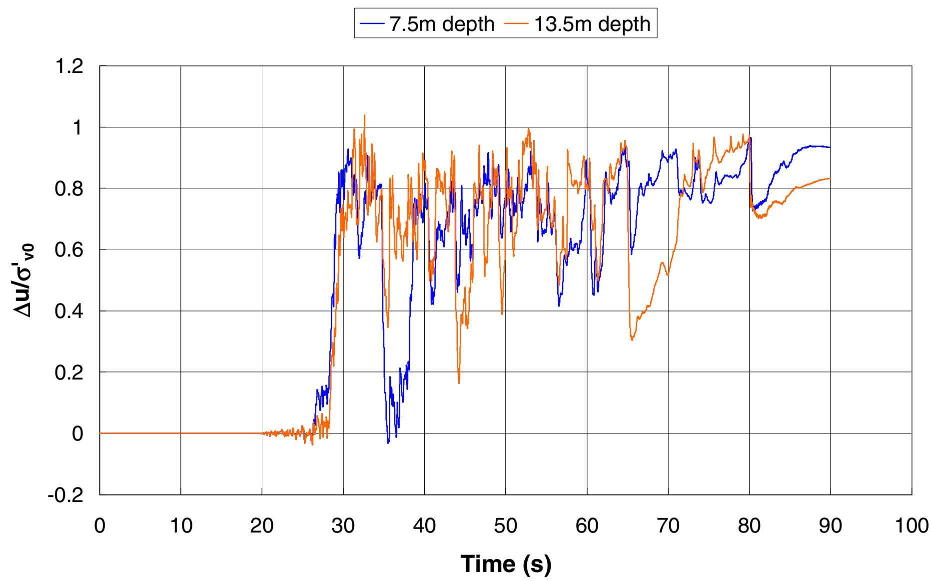

Fig. 4: Evolution of the pore-fluid over-pressure ratio computed at 7.5m and 13.5m depths (two-phase non linear simulation)

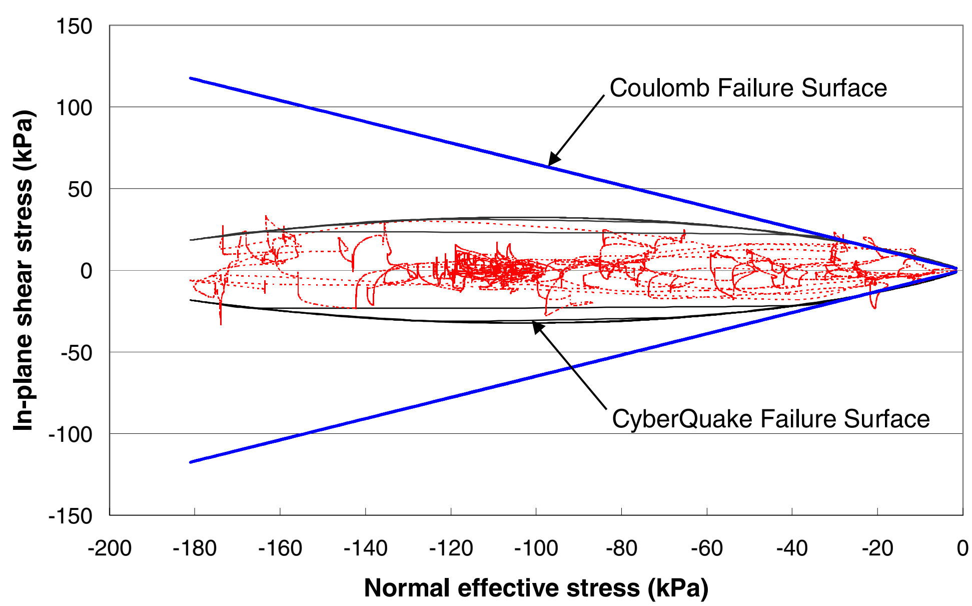

Fig. 5: In-plane stress diagram computed at 13.5m depth (two-phase non linear simulation)

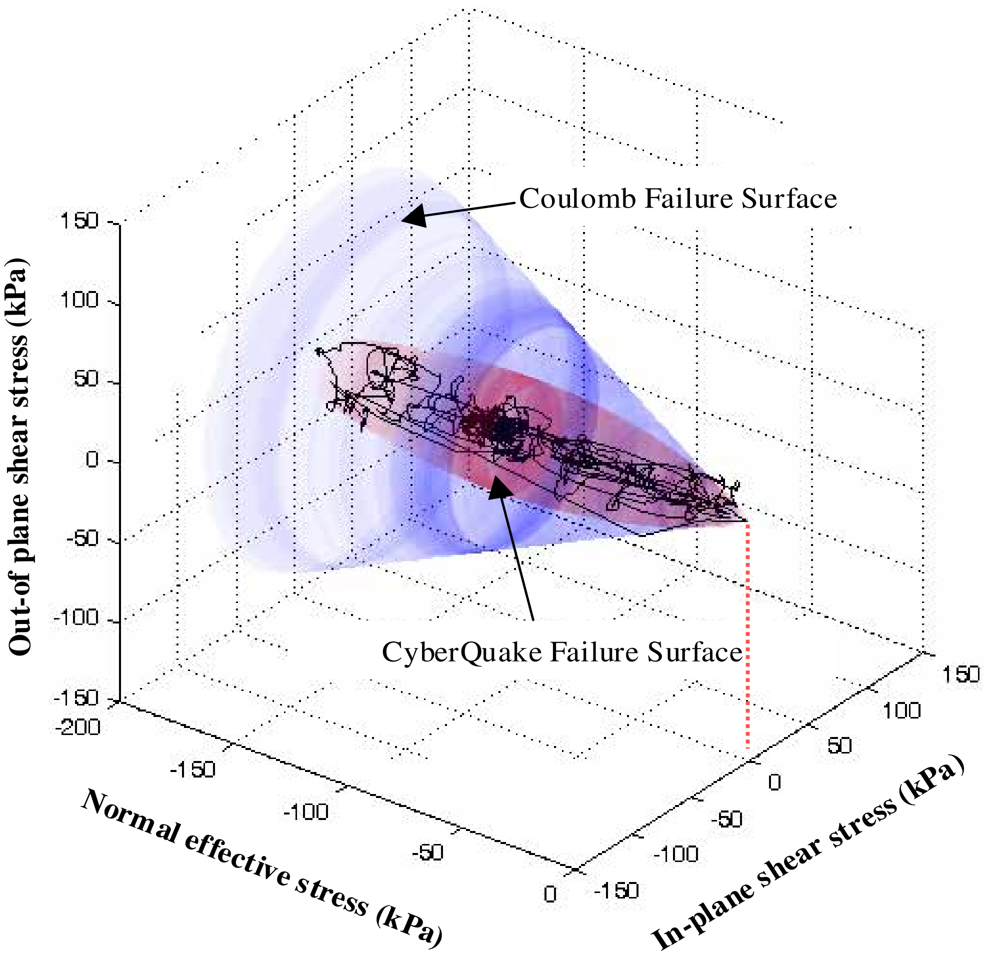

Fig. 6: 3D representation of shear stresses vs. normal effective stress at 13.5m depth (two-phase non linear simulation)

A comparison between the initial, measured and computed post-seismic shear wave velocity (Vs) profiles is also presented on Figure 7. We see that for the Vs profile computed after the earthquake, a strong reduction occurs between 0 and 9m, which corresponds to the expected liquefaction mechanism in this range. On the contrary, the post-earthquake field data shows a significant increase between 2.5 and 8m (Lee [6]). The author suggests this is due to the consolidation process and rearrangement of the particles after liquefaction, mechanisms which are not accounted for in *CyberQuake* at present.

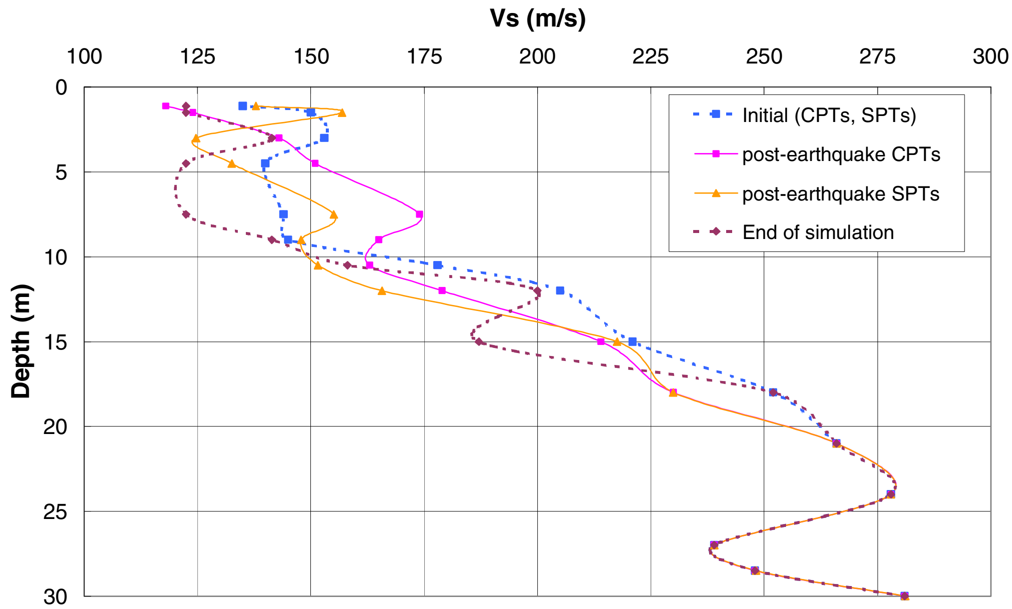

Fig. 7: Initial, measured (SPTs, CPTs) and computed post-earthquake V_s profiles

Figure 8 shows the evolution of both shear modulus (G) and pore-fluid over pressure computed at 7.5m depth during the seismic motion. What is interesting to note on this figure is the direct link between pore-pressure build-up and G reduction on one hand (liquefaction process), and between large drop of pore-pressure and G increase on the other hand, attesting a recovering of shear resistance within the soil layer and resulting from dilatancy.

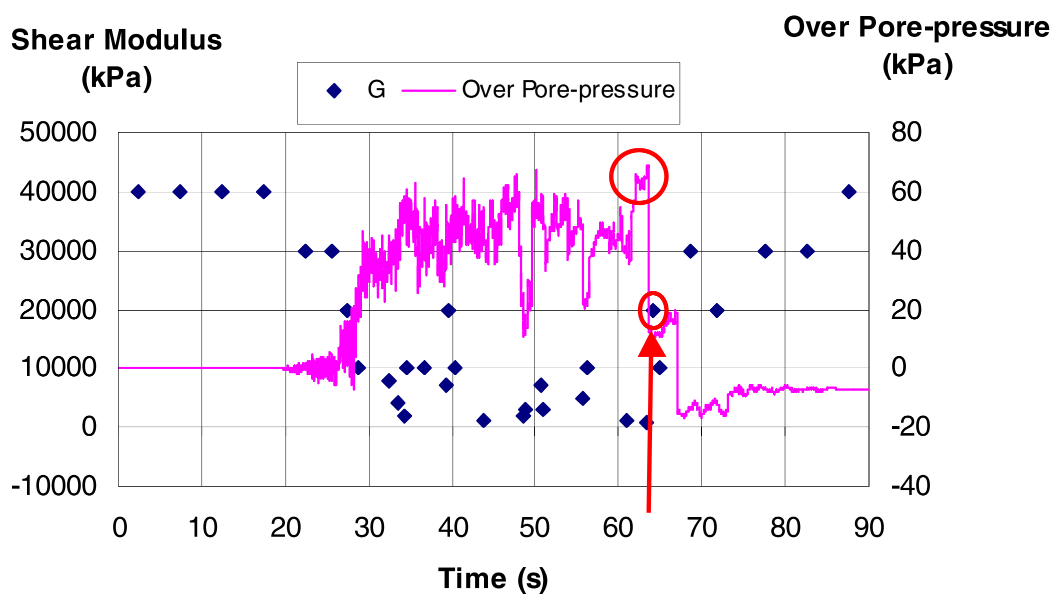

Fig. 8: Evolution of the shear modulus and pore-fluid over pressure at 7.5m depth (two-phase non linear simulation)

CONCLUSION

The non linear site effect analysis presented on this paper was conducted for a site located within the Chang-Hwa Coastal Industrial Park during the 1999 Chi-Chi earthquake in Taiwan. Computed NS, EW and UP ground accelerations obtained with *CyberQuake* model under undrained and two-phase assumptions, are compared with the corresponding accelerations recorded at seismic station TCU117, a station located on a soft soil about 3 km from the studied site. Amplitudes, peak location and frequency contents computed with the two-phase assumption are closer to the recorded data. Moreover, both non linear simulations and especially the two-phase one, are able to reproduce the liquefaction state (shear modulus or Vs reduction) observed for the saturated sandy layers in the 1.3-13.5m depth range. This range is in good agreement with the top 14m highlighted by the field campaign of penetration tests carried out after the earthquake. Finally, a maximum vertical ground displacement of 20cm was computed as well, value to be compared to the 33-45cm recorded in the area after the earthquake.

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