

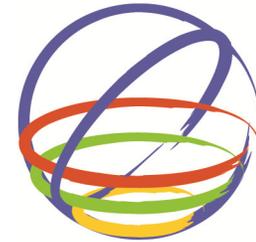
# Integrated foundation-structure modelling of a case study from the Christchurch 2011 earthquake

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## **SUMMARY:**

During the reconnaissance work carried out following the 22 February 2011 earthquake, many high value buildings have been identified to be severely affected by nonlinearities that occurred at the foundation level. Among them, a ten-storey reinforced concrete frame affected by differential settlement and characterized by weak beam-strong column seismic response has been selected for in-depth numerical investigations. The two-dimensional structural response under lateral loading has been simplified as a single-degree-of-freedom system through quasi-static analyses. The dynamic response of the simplified model with both fixed and nonlinear flexible bases has been investigated under the recorded time-history. The results have been critically discussed and compared against the observed residual deformations. The numerical findings confirm that the nonlinear soil-foundation interface behaviour is a key modelling component. It has a critical influence on the overall structural behaviour and performance and changes the mechanism of failure beyond the conventional expectations.

*Keywords: Soil-structure-interaction, modelling, macroelement, Christchurch earthquake*

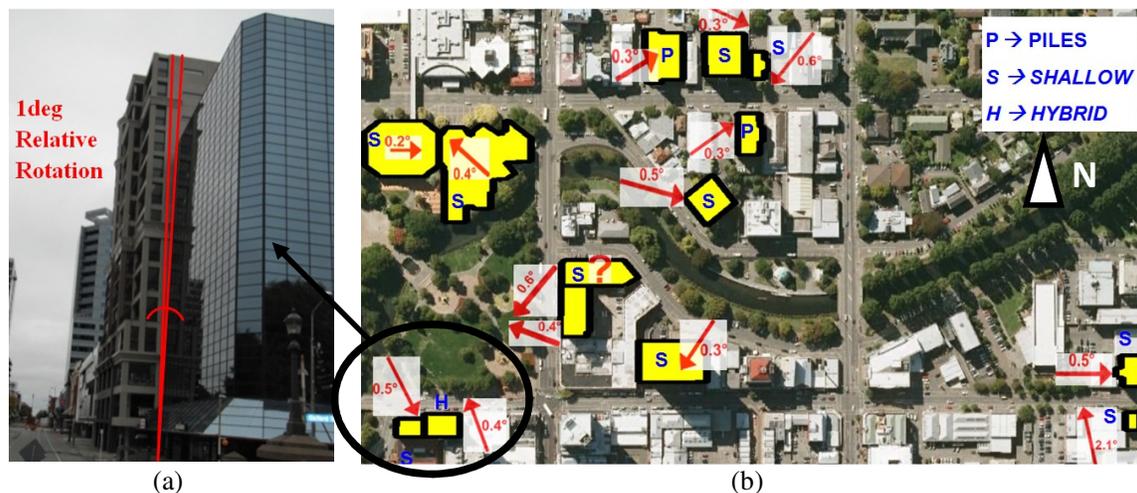
## **1. INTRODUCTION**

During strong ground shaking, both the soil and structure exhibit highly nonlinear behaviour resulting in large permanent deformations and residual stresses. While structures are commonly designed to take advantage of their ductility to achieving target performances, the soil is usually considered to remain in the elastic domain. The flexibility of the soil-foundation interface is neglected or at most considered as equivalent linear (Stewart et al., 2003).

Material and geometric nonlinearities at the foundation level may occur, resulting in energy dissipation and foundation uplift. Recent earthquakes (e.g. Kocaeli, 1999 and Christchurch, 2011) have shown that the occurrence of the above mentioned phenomena due to soil yielding or mechanical property degradation, i.e. liquefaction, might lead to punching or differential settlement and lateral movement of the structure (Cubrinovski et al., 2011). For example, Figure 1b shows the partial plan view of the central business district (CBD) of Christchurch, with the location of selected investigated buildings and their residual tilt. It can be noticed that many buildings (most of them located where the soil liquefied) have been affected by differential settlement, regardless the foundation type they sit on. It is envisaged that, the nonlinearity that took place at the soil level decreased the ductility demand on the superstructure and thus the damage. The beneficial role of soil nonlinearity on the seismic demand of superstructures has also been confirmed by some numerical studies (Pecker and Chatzigogos, 2010; Moghaddasi et al., 2012; Grange et al., 2009).

As “the strength of a chain is the strength of its weakest link” (Paulay and Priestley, 1992), the soil performing in nonlinear range could become the fuse in the load path. This would result in a change of strength hierarchy; the structure would not respond as it would be expected to do, but could be partially ‘saved’ by the energy dissipation occurring in the soil. However, despite the reduction of

superstructure damage, large and sometimes not negligible residual deformations may develop as a consequence of this soil energy dissipation process. This becomes of little structural value, unless the phenomenon is controlled during the design phase.



**Figure 1.** Residual tilts observed in the Christchurch CBD following the February earthquake: (a) Location and relative rotation of the selected case study with respect to the next door building; (b) Selected investigated buildings along with their foundation types

In light of these observations and given the amount of dissipation capacity potentially available in the soil, current research moves towards new design philosophies, in which the soil is allowed to yield under rocking or sliding of foundations (Gazetas, 2006; Anastasopoulos, 2009; Toh and Pender, 2009). However, before these concepts are implemented in design practice, reliable and relatively simple numerical tools are needed to properly simulate the interaction occurring between soil, foundation and structure during a seismic event. The uplift and soil plasticity, in addition to the displacement demand on the superstructure caused by foundation translation and rotation, will in turn change the dynamic load on the soil. This leads to a sequence of soil-foundation-structure-foundation-soil interaction.

Simplified models able to simulate the nonlinearity at the soil-foundation interface include the beam-on-nonlinear-Winkler foundation (BNWF) (e.g. Harden and Hutchinson, 2009), and the macroelement formulation (e.g. Grange et al., 2009; Gajan and Kutter, 2009; Chatzigogos et al., 2011). The former is an assembly of bilinear springs and dampers where gapping is allowed; the latter is a single element characterized by a nonlinear constitutive law, able to link the generalized force parameters to the corresponding kinematic ones and to take into account the coupling occurring between moment, shear and axial responses.

This study investigates soil-structure-interaction (SSI) effects considering nonlinearities occurring at the soil-foundation (SF) interface. Particular emphasis is given to the change in ductility demand to the superstructure due to the flexibility in the nonlinear range of the soil. Reconnaissance work carried out following the 22 February 2011 Christchurch earthquake (Cubrinovski et al., 2011; Giorgini et al., 2011), identified a building as apparently affected by differential settlement (Figure 1a). It has therefore been selected for in-depth numerical investigations. The response of the fixed and the nonlinear flexible base structures are compared when subjected to one of the recorded time-histories from this event. The nonlinearity at the SF interface is taken into account using the macroelement formulation developed by Chatzigogos et al., 2009) and implemented in Ruaumoko 3D (Carr, 2010). The numerical simulations are also compared against the observed residual deformations. This work has not to be considered as an exhaustive explanation of the actual seismic response of the building, but as a first attempt in understanding its performance during the Christchurch earthquake.

The foundation-structure system, available soil information at the site and the damage observed following the Christchurch earthquake are described in Section 2. The methodology and the results of the performed time-history analyses are presented in Section 3.

## 2. BUILDING DESCRIPTION, SOIL INFORMATION AND OBSERVED DAMAGE

The building is highlighted with a circle in Figure 1b and refers to the glazed building located on the left side of the plan.

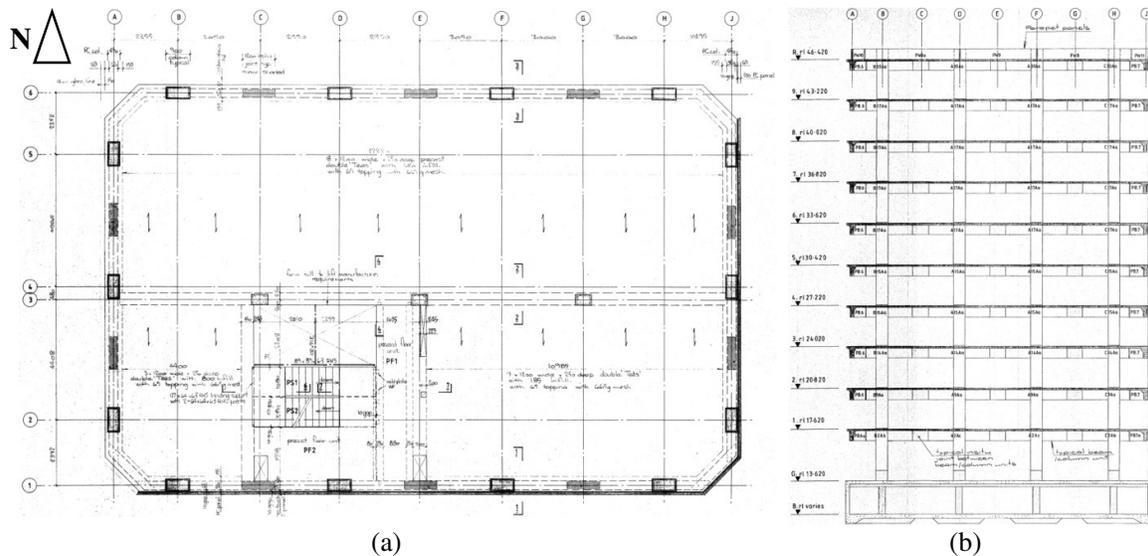
### 2.1. Structure-foundation system

The selected structure is a 10 storey reinforced concrete building built in 1986. The partial original drawings are reported in Figure 2 and Figure 3.

The seismic resistant structural system comprises of perimeter precast frames in both north-south and east-west directions (grids A, J and grids 6, 1 respectively). The precast units are the beam-column joints, with in situ connections typically located at mid-span. The upper floors are 250mm deep double tees. The structural layout is very regular, with two three-span frames along the east-west direction and two two-span frames along the north-south direction.

The gravity resistant structural system is the same as the seismic one, but a third frame is added in the east-west direction (grid 3). The floor units are simply supported and they span in north-south direction.

The building sits on a 2.6 metres embedded basement. Footings are located beneath the columns; they are connected through a 300mm deep concrete slab.

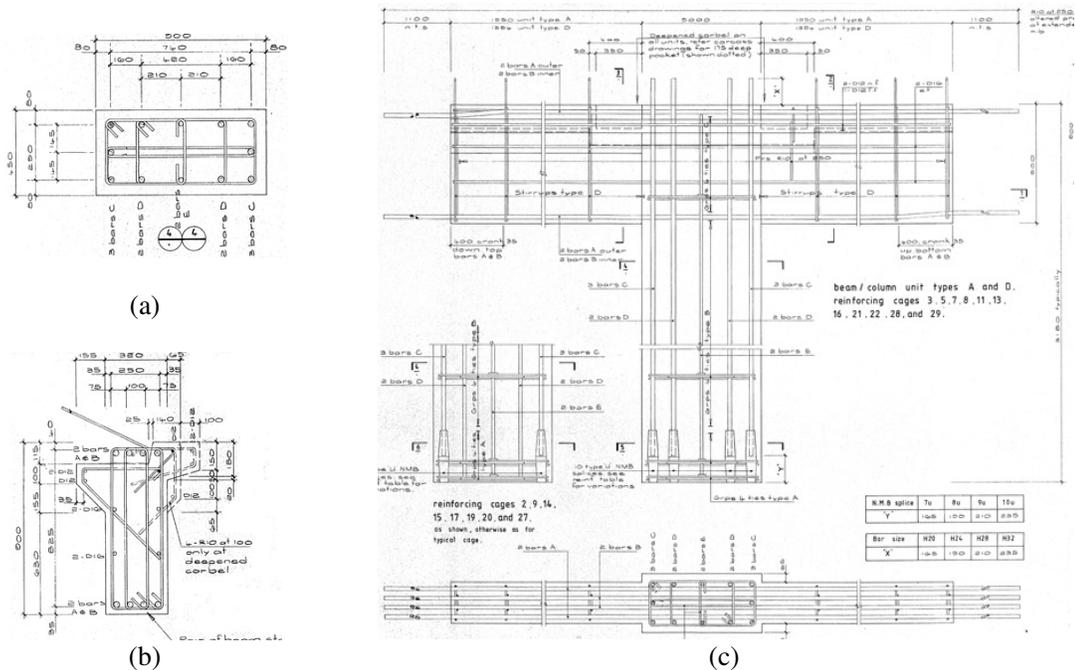


**Figure 2.** Original drawings of the selected case study 10 storey reinforced concrete building:  
(a) Typical structural plan; (b) Elevation (E-W direction)

### 2.2. Soil information

The shallow alluvial soils beneath the Christchurch CBD are very variable within short distances, both horizontally and vertically. Therefore, although following the earthquake severe liquefaction has been observed in some areas, any liquefaction manifestation has not been noticed in others. More

information about the CBD soil profiles and the location of observed liquefaction can be found in Cubrinovski et al., 2011. The structure is located in a zone of transition between shallow gravel and liquefiable soil. The geotechnical report relative to the adjacent site (McCahon, 2004), describes a not liquefiable soil profile, with the exception of confined lenses of loose sand immediately underlying the gravel at about 10m depth.



**Figure 3.** Original drawings of the selected case study RC 10 storey building: (a) Typical column section; (b) Typical beam section; (c) Precast beam-column connection unit

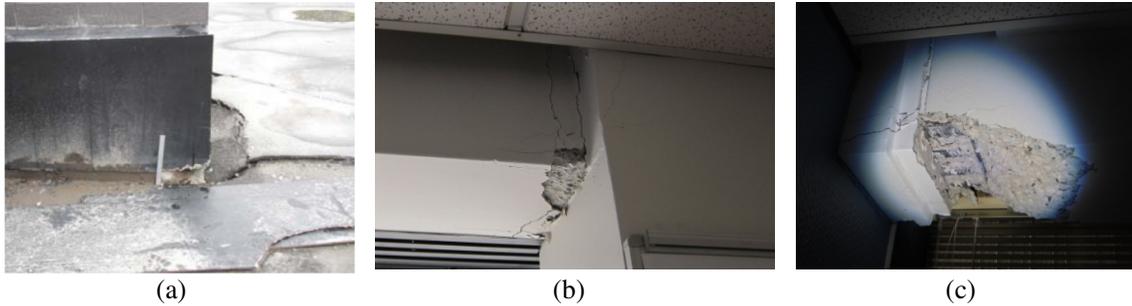
### 2.3. Observed Damage

Figure 1b shows that the two buildings located next to each other have a residual tilt of opposite directions.

Little evidence of liquefaction was observed around the building; however, sand boils up to 200mm deep were identified in the park on the other side of the road. As shown in Figure 1b, the measured residual tilt of the building was about 0.5 degrees south-east. Non-uniform settlement was observed along the perimeter. The distortion was localized at the two corners along the east-west direction (Figure 4a shows the eastern corner). This asphalt movement suggests a “rocking-dominated” motion of the building, excited by an east-west earthquake directivity.

Figure 4b and c illustrate the structural distress. The building revealed a weak beam-strong column type of response, with widespread plastic hinges in the beams of most of the floors (internal inspection was partial). This is consistent with the design philosophy in place since the mid 1970s, which included considerations of capacity design principles and design for a beam-sway mechanism.

Overall, the building seemed to have experienced a combined soil-foundation-structure type of response, resulting in permanent residual deformations from both structure and soil deformations. Complications that were speculated on during the inspections were building to building interaction through the soil along with some possible soil non-uniformity (Giorgini et al., 2011). However, these are neglected in this study.



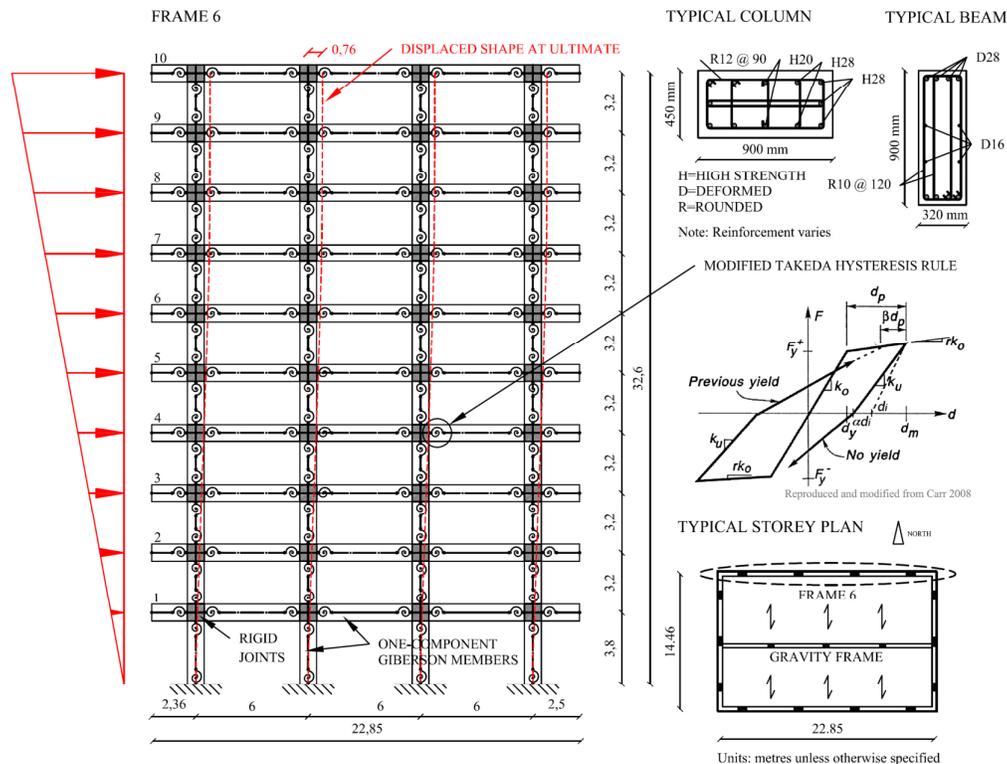
**Figure 4.** Observed damage in the selected case study: (a) Relative movement soil-to-building; (b) and (c) Beam plastic hinges (Courtesy of W. Y. Kam)

### 3. MODELLING AND TIME-HISTORY ANALYSES

#### 3.1. Structural modelling

As the Christchurch earthquake had a predominant east-west directivity (Kam et al., 2011), the analyses focus on one of the seismic frames running in that direction (grid 6, Figure 2).

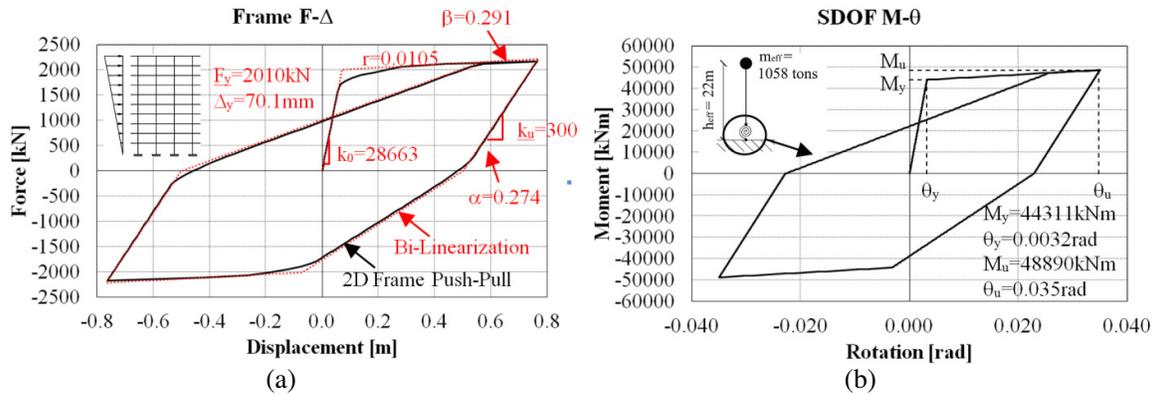
A push-pull analysis has been performed in order to find the force-displacement response of the frame under quasi-static loading. Figure 5 summarizes the features of the adopted numerical macro-model, based on a lumped-plasticity approach, along with the typical sections of both beams and columns (reinforcement details vary and are not specified).



**Figure 5.** Two-Dimensional numerical macro-model of the frame system along grid 6

The structural members have been modelled as one-component Giberson elements and characterized by the stiffness-degrading modified-Takeda hysteretic rules. The moment-curvature diagram of each member has been defined using the Matlab code Cumbia (Montejo and Kowalsky, 2007), taking into account both steel strain hardening and concrete confinement (Mander et al., 1988). Rigid-end blocks have been considered at each beam-column joint. The force-distribution in the push-pull analysis has been assumed as triangular.

Figure 6a shows the force-displacement diagram of the 2D frame, which has been linearized and calibrated to fit a modified Takeda type rule. The latter, changed to an equivalent moment-rotation relationship, has been applied to an equivalent single-degree-of-freedom (SDOF) system having lumped plasticity at the bottom of the cantilever (Figure 6b). The SDOF system characteristics, i.e. equivalent mass  $m_{eff}$  and equivalent height  $h_{eff}$ , have been defined according to the displacement-based-design (DBD) procedure (Priestley et al., 2007); however, instead of the normalized displaced shape, the actual deformed configuration resulted from the push-pull analysis on the frame has been used. The seismic weight has been calculated in accordance with the NZS 1170.5, 2004. The characteristics of the SDOF system are summarized in Figure 6b.



**Figure 6.** (a) Force-Displacement relationship of the modelled frame along with the adopted bi-linearization; (b) Depicted Moment-Rotation relationship of the equivalent SDOF system used for non-linear history analysis

### 3.2. Soil-foundation modelling

The SDOF system sketched in Figure 6b has been used for non-linear time histories analyses in both the fixed and the nonlinear flexible base configurations.

The soil-foundation interface parameters are listed in Table 1 and Table 2. The reader is referred to the original work of Chatzigogos et al., 2009, for the full model formulation. The soil beneath the case study is not a purely cohesive soil (an assumption for the considered macroelement). Therefore, an equivalent undrained cohesion has been defined through the calculation of the shear strength corresponding to the vertical stress in the Mohr-Coulomb diagram ( $\phi = 35 \text{ deg}$ ).

**Table 1.** Mechanical and elastic properties of the soil-foundation interface

Soil-Foundation elastic properties							
Soil Weight	$\gamma_s$	20	$\text{kN/m}^3$	Foundation rotational weight	$w_{f,r}$	7045	$\text{kNm}^2$
Poisson's ratio	$\nu$	0.3	/	Vertical stiffness	$K_v$	1017174	$\text{kN/m}$
Soil shear wave velocity	$V_s$	135	$\text{m/s}$	Horizontal stiffness	$K_h$	837673	$\text{kN/m}$
Soil shear modulus	$G_0$	37411	$\text{kN/m}^2$	Rocking stiffness	$K_r$	15352405	$\text{kNm}$
Soil cohesion	$c_0$	78	$\text{kN/m}^2$	Radiation damping (vertical)	$C_v$	70195	$\text{kNs/m}$
Foundation radius	$r_{eq}$	4.76	$\text{m}$	Radiation damping (horizontal)	$C_h$	45402	$\text{kNs/m}$
Foundation weight	$w_{f,h}$	1245	$\text{kN}$	Radiation damping (rocking)	$C_r$	397299	$\text{kNms}$

Note: Refer to Gazetas, 1991, for the calculation formulas of the elastic parameters

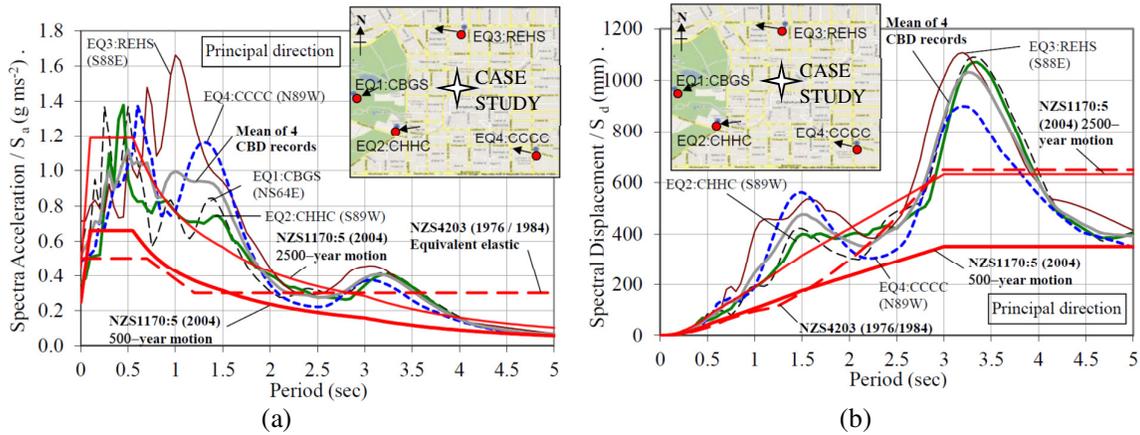
To be noticed that the radiation damping has been defined using far field soil properties ( $V_s=313\text{m/s}$ ).

**Table 2.** Plasticity and uplift parameters of the macroelement model

Maximum centred vertical force	$N_{\max}$	33822	kN
Maximum normalized horizontal force	$Q_{V\cdot\max}$	0.165	/
Maximum normalized moment	$Q_{M\cdot\max}$	0.11	/
Numerical parameter expressing the extent of the initial plastic stiffness	$p_1$	0.5	/
Numerical parameter expressing the extent of stiffness degradation in reloading	$p_2$	4	/
Uplift parameter (uplift moment)	$\alpha$	6	/
Uplift parameter (uplift moment)	$\beta$	2	/
Uplift parameter (rocking stiffness after uplift)	$\gamma$	2	/
Uplift parameter (rocking stiffness after uplift)	$\delta$	0.5	/
Uplift parameter (coupling between horizontal and rocking stiffness after uplift)	$\epsilon$	0.2	/
Uplift-plasticity coupling parameter	$\zeta$	1.5	/
Limit ratio for toppling	$Q_M/Q_{M,0}$	3	/

### 3.3. Time-history analyses and results

Figure 7a and b reports the elastic acceleration and displacement response spectra, together with the indication of the recording station locations with respect to the selected case study. The Christchurch Hospital (CHHC) record has been selected for the numerical analyses. The selection criterion was based on the soil similarity. The time history recorded along the principal direction (east-west) is shown in Figure 8a; the maximum PGA is 0.38g.



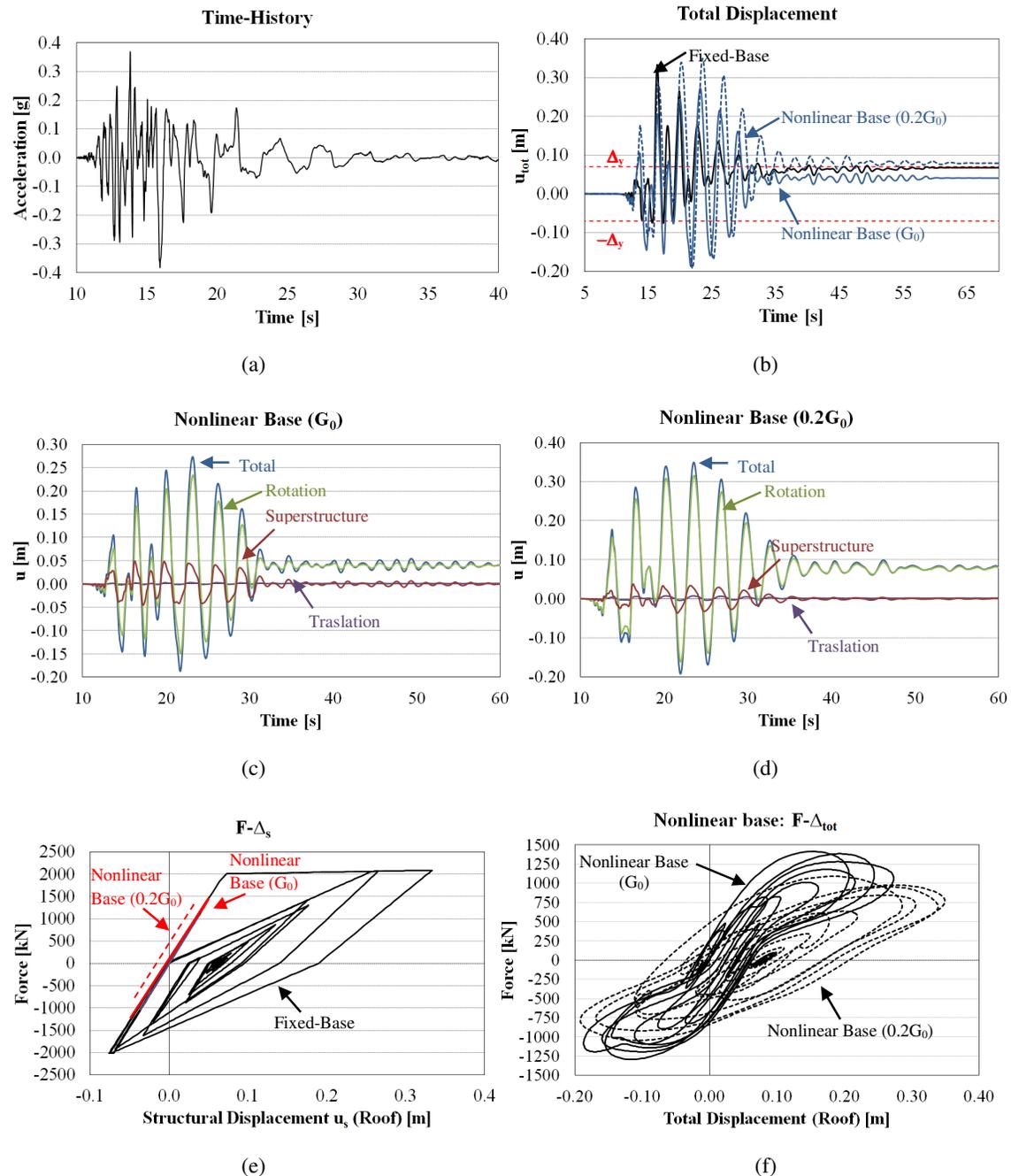
**Figure 7.** 5%-damped elastic response spectra in the Christchurch CBD along the principal horizontal direction (E-W) and comparison with the NZS1170:5 design spectra (red solid) for Christchurch (soil class D,  $R=10\text{km}$ ): (a) Acceleration spectrum; (b) Displacement spectrum (Kam et al., 2011)

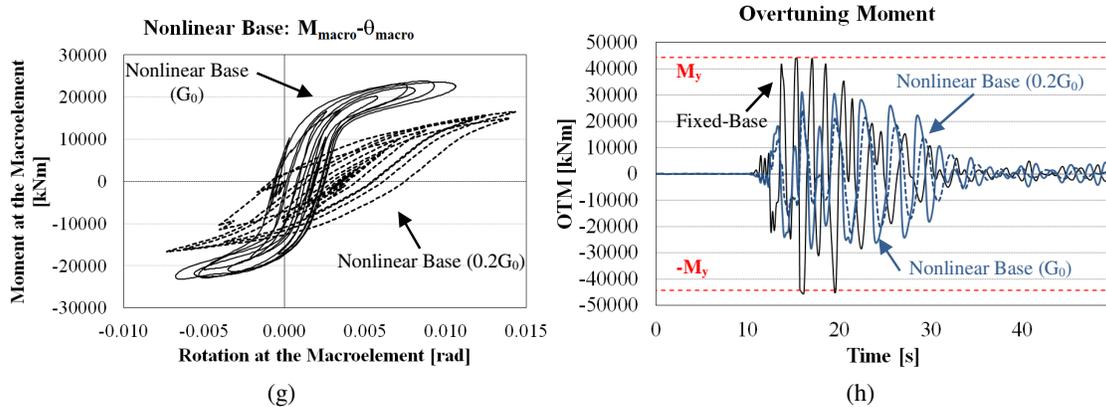
The main vibration periods of the fixed and nonlinear flexible base systems in the numerical model are 1.21 and 1.68 seconds respectively (the latter is longer as expected from the increased flexibility of the system). The effect of soil stiffness degradation on the seismic response is also evaluated. A lower bound of 20% residual stiffness is considered. It causes an increase of the natural period up to 2.89 seconds.

The analysis results are reported in Figure 8. The total displacement measured at the top of the cantilever in the flexible based model shows more significant cycles with respect to the fixed-base model (Figure 8b). Also, as expected, the displacement rate increases as the soil stiffness decreases.

Within the amount of total displacement in the flexible base models, the contributions of the

superstructure deformation and the rigid translation and rotation of the foundation can be separated. This is shown in Figure 8c and d, where it can be noticed that the foundation rocking is dominant (relative to the total displacement), while the superstructure contribution is minor. This is consistent with the force-displacement relationships. For instance, while the superstructure in the fixed-base model yields, any nonlinear structural behaviour is not shown when the macroelement is considered (Figure 8e). In the flexible base model, the energy dissipation occurs at the soil-foundation interface. This is shown in Figure 8f and g, which respectively illustrates the total force-displacement diagram of the integrated system and the moment rotation relationship of the macroelement. As the soil stiffness decreases, larger deformations are reached with smaller forces, defining a slightly fatter loop. When soil nonlinearity is considered, the overturning moment experienced by the cantilever is significantly smaller, especially when the soil is characterized by degraded shear modulus (Figure 8h).





**Figure 8.** Numerical results: (a) Time-History; (b) Total displacement at the roof level; (c) (d) Enlargement of the different contributions within the total displacement due to foundation nonlinearities; (e) (f) Force-displacement relationships in terms of structural and total top displacement; (g) Macroelement moment-rotation diagram; (h) Overtuning moment.

The displacement reached in the fixed-base model (Figure 8b) is well above the first yielding of the frame (located at about 46 mm), and would suggest a displacement ductility demand of 4.76. However, this does not agree with the damage observed following the earthquake (Figure 4b and c). Also, although similar displacement values are reached when the macroelement with the reduced shear modulus is considered, they are not a good indication of the actual structural ductility demand.

The overestimation of ductility demand in the fix-based model is possibly due to the lack of modelling of energy dissipation in the soil, which has possibly had a beneficial effect on the seismic demand acting on the superstructure.

From the soil residual deformations point of view, the fix-based model does not agree as well with what has been observed in the field. The differential settlement is smaller than the observed tilt along the east-west direction (0.3 degrees east, which corresponds to a top displacement of around 170 mm). Further degradation of the soil properties could have occurred because of development of pore water pressure and/or the presence of non-uniform soil profiles beneath the building. Also, the asymmetry of the loads transferred from the frame to the foundation could have had a crucial role.

#### 4. CONCLUSIONS

A 10 storey RC building designed in the 1980s and located in the Christchurch CBD has been numerically investigated under some of the recorded time-histories during the February 2011 earthquake. Overall, the numerical results have confirmed the important role played by nonlinear SSI in the seismic response of the building. In fact, the overestimation of displacement ductility demand in the superstructure suggests a not negligible amount of energy dissipation at the foundation level.

However, the limitations of the adopted modelling approach and of the initial assumptions have been identified. On one side, it is expected that multi-degree-of-freedom (MDOF) modelling approach for the superstructure would capture the actual distribution of internal actions and the consequent non uniform load transferred to the foundation. On the other side, more local modelling of the foundation with discrete macroelements underneath each column would be able to capture the induced differential settlement due to soil non uniformity. Further research will address the mentioned issues.

The use of simplified tools (such as the macroelement herein adopted), able to take into account uplift and soil plasticity together with the inertial interaction arising from the structure, is very promising for the analysis of MDOF systems where the above mentioned phenomena are critical for predicting the actual response.

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