

Pre and post-earthquake dynamic analysis of an RC building including soil-structure interaction

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ABSTRACT: This paper presents a study of the dynamic analysis of a building before and after devastating earthquakes using system identification. The eight storey reinforced concrete building is instrumented with 10 accelerometers. The time domain N4SID system identification technique was used to obtain the frequencies, damping ratios and mode shapes considering fixed and pseudo flexible base models taking into account the soil-structure-interaction. The response of the building before and after the Darfield (04/09/2010) and Christchurch (22/02/2011) earthquakes was ascertained to evaluate the significant changes in the behavior. It was concluded from the investigation that monitoring the seismic response of the building is useful for a better understanding of its behaviour during earthquakes, and also that the participation of soil is significant towards the seismic response of the building and it should be considered in models to simulate the real behavior.

1 INTRODUCTION

The full scale, in-situ investigations of instrumented buildings present an excellent opportunity to observe their dynamic response in as-built environment, which includes all the real physical properties of a structure under study and its environment. These studies are useful for the improvement of methodologies involved in e.g. design and analyses of structures, model updating and structural health monitoring. Researchers are using system identification techniques to extract dynamic properties of structures from the recorded responses of ambient or forced excitations (De Roeck et al. 2000; Skolnik et al. 2006). A major portion of the system identification procedures are concerned with computing polynomial models which are known to give rise to ill-conditioned mathematical problems. This is mostly the case for multiple-input/multiple-output systems. Numerical algorithms for subspace state-space system identification (such as N4SID) can perform better, especially for higher order systems (Van Overschee & De Moor 1994). Another major advantage is their non-iterative nature, which guarantees convergence, insensitivity to initial estimates and absence of local minima of the objective function. It is therefore recommended that such procedures should be followed in the analysis to provide reliable solutions. N4SID is considered to be one of the most powerful classes of known system identification techniques in the time domain (Van Overschee & De Moor 1994) and is, therefore, used in the present study for estimating frequencies, damping ratios and mode shapes.

A structure, its foundation and the surrounding soil constitute a system of dynamically interacting parts. Due to the flexibility of soil, the system period can be longer than the period of the fixed base building. Building period constitutes an important part in the design and analysis of earthquake resistant structures. Soil-structure-interaction investigations, therefore, are necessary to better understand the actual response of structures during earthquakes. The objective of this study is to assess and understand the seismic response of the instrumented building before and after the devastating Darfield (04/09/2010) and Christchurch (22/02/2011) earthquakes to ascertain the behaviour of the building system (soil-foundation-structure) based on the frequency versus response levels. In particular, trends between peak response acceleration (PRA) and the identified first three frequencies of the building will be investigated using nine recorded seismic events. This paper will thus firstly describe the building and sensor array, secondly the methodology detailing the N4SID system identification technique with a simple model to evaluate the SSI using system identification, and

finally the analysis of the seismic response of the building describing selection of earthquakes, modal system identification results and the evaluation of the difference in the behaviour of the building before and after the strong events including effects of SSI.

2 DESCRIPTION OF THE BUILDING AND SENSOR ARRAY

The building studied is an eight storey RC building made up of 21 moment resisting frames positioned at 2.74m centres along the longitudinal axis of the building as shown in Figure 1. In addition to the frames the building is stiffened with three shear walls standing near both ends and in the middle along the length of the building. The building is 58.8m long and 14.9m wide with inter-storey height of 4.32m and the foundation system consists of shallow pad type footings.

This building was instrumented in 2006 and 10 sensors were installed at different floors and ground levels as shown in Figure 2. Initially, three tri-axial and one vertical sensor at the ground level, two sensors (one vertical and the other bi-axial) at level 4, two bi-axial sensors at level 7 and two vertical sensors at level 8 were installed. Later on, this sensor array was upgraded and all the vertical and biaxial sensors were replaced by tri-axial sensors except sensor 24 and 28 at the ground level and level 7 respectively. This building does not have a free-field sensor.

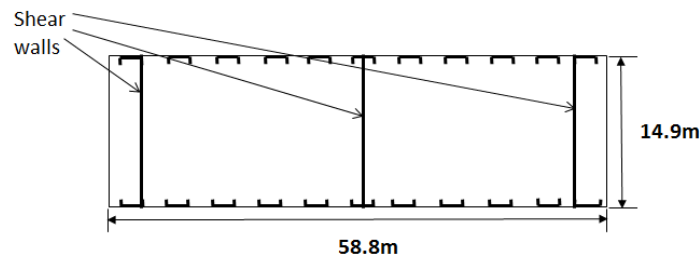


Figure 1. Sketch of typical floor plan of the building showing location of shear walls

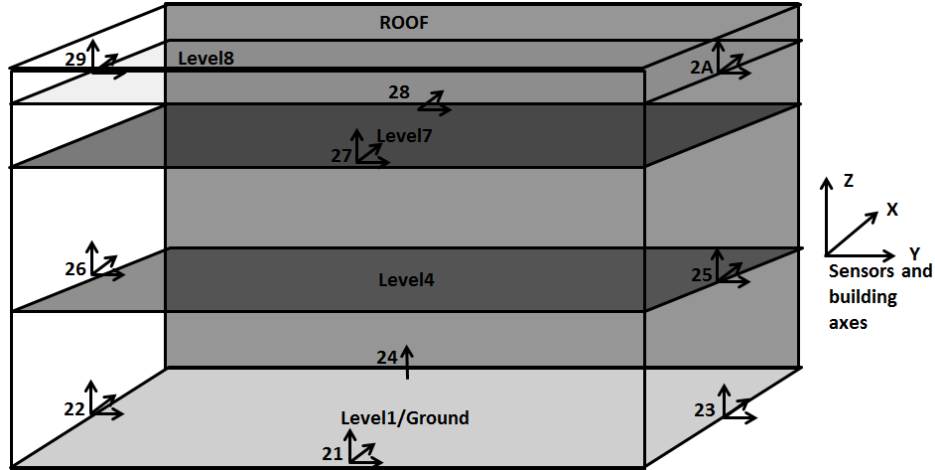


Figure 2. Location of sensors within the building

3 METHODOLOGY

3.1 N4SID system identification technique

This section provides a brief explanation of the N4SID system identification technique. Full details of the technique can be found in Van Overschee and De Moor (1996). After sampling of a continuous time state space model, the discrete time state space model can be written as:

$$\mathbf{x}_{k+1} = \mathbf{A}\mathbf{x}_k + \mathbf{B}\mathbf{u}_k + \mathbf{w}_k \quad (1)$$

$$\mathbf{y}_k = \mathbf{C}\mathbf{x}_k + \mathbf{D}\mathbf{u}_k + \mathbf{v}_k \quad (2)$$

where \mathbf{A} , \mathbf{B} , \mathbf{C} and \mathbf{D} are the discrete time state, input, output and control matrices respectively,

whereas \mathbf{x}_k and \mathbf{y}_k are the time state and output vectors, and \mathbf{u}_k is the excitation vector, respectively. Vectors \mathbf{w}_k and \mathbf{v}_k are the process and measurement noise, respectively, that are always present in real-life applications. In case of input/output system identification, data from both output \mathbf{y}_k and input \mathbf{u}_k are assembled in a block Hankel matrix, which is defined as a gathering of a family of matrices that are created by shifting the data matrices in time. After this, the identification involves two steps. The first step takes projections of certain subspaces calculated from input and output observations (in the block Hankel matrix) to estimate the state sequence of the system. This is usually achieved using singular value decomposition and QR decomposition. In the second step, a least square problem is solved to estimate the system matrices \mathbf{A} , \mathbf{B} , \mathbf{C} and \mathbf{D} . Then modal parameters, i.e. natural frequencies, damping ratios and mode shapes, are found by eigenvalue decomposition of the system matrix \mathbf{A} . The analyst, however, has to determine a proper system order. The approach based on observing trends of the estimated modal parameters in the so-called stabilization charts (Figure 3) is often used: a range of system orders is tried and modal parameters which repeat themselves across that range are accepted as correct results. Stability tolerances are chosen based on the relative change in the modal properties, i.e. modal frequencies, damping ratios and mode shapes, of a given mode as the system order increases. For mode shapes stability, model assurance criterion (MAC) between the mode shapes of the present and previous orders were examined. MAC is an index that determines the similarity between two mode shapes. For modes ϕ_i and ϕ_j , the MAC is defined as (Ewins 2000):

$$MAC = \frac{(\phi_i^T \phi_j)^2}{(\phi_i^T \phi_i)(\phi_j^T \phi_j)} \quad (3)$$

In Equation (3), superscript T denotes vector transpose.

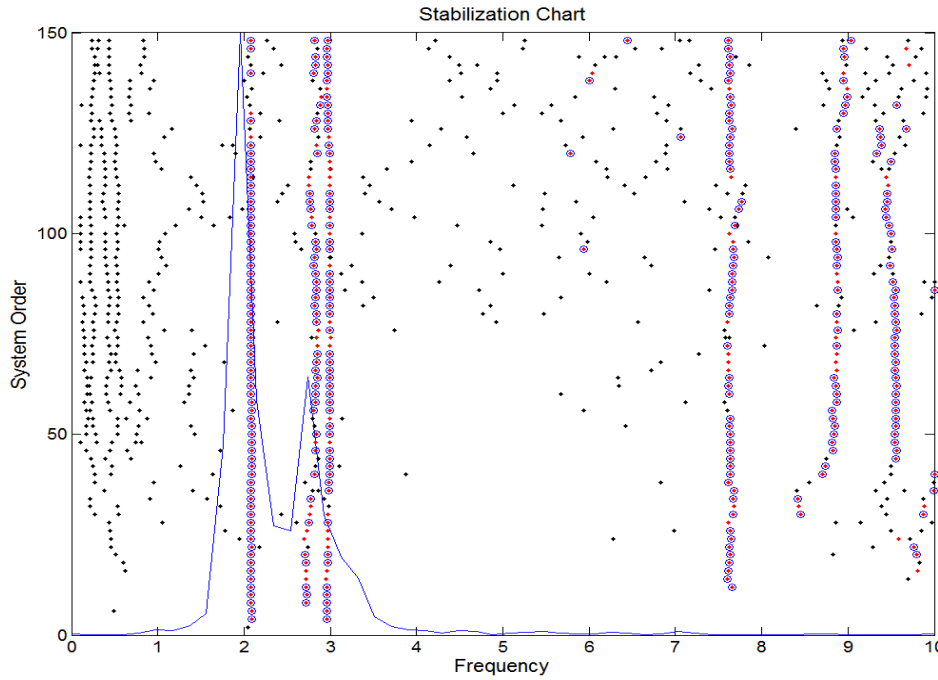


Figure 3. A typical stabilization chart (superimposed by power spectral density curve) showing trends of frequencies

3.2 System identification for evaluating SSI effects

For evaluation of SSI effects using system identification procedures, Stewart and Fenves (1998) proposed an approach which is followed in this study. Consider the structure shown in Figure 4. The height h is the vertical distance from the base to the roof (or another measurement point located on the building). The symbols denoting translational displacements are as follows: u_g for the free field translational displacement, u_f for the foundation translational displacement with respect to the free field, and u for the roof translational displacement with respect to the foundation. Foundation rocking angle is denoted by θ , and its contribution to the roof translational displacement is $h\theta$. The Laplace

domain counterparts of these quantities will be denoted as \hat{u}_g , \hat{u}_f , \hat{u} and $\hat{\theta}$, respectively.

Stewart and Fenves (1998) consider three different models and associated transfer functions (H_1 , H_2 and H_3) as follows:

- Flexible base model

$$H_1 = \frac{\hat{u}_g + \hat{u}_f + \hat{u} + h\hat{\theta}}{\hat{u}_g} \quad (4)$$

where input is the free field displacement u_g and output is the total roof displacement $u_g + u_f + u + h\theta$.

- Pseudo flexible base model

$$H_2 = \frac{\hat{u}_g + \hat{u}_f + \hat{u} + h\hat{\theta}}{\hat{u}_g + \hat{u}_f} \quad (5)$$

where input is the total foundation translational displacement $u_g + u_f$ and output is the total roof displacement $u_g + u_f + u + h\theta$.

- Fixed base model

$$H_3 = \frac{\hat{u}_g + \hat{u}_f + \hat{u} + h\hat{\theta}}{\hat{u}_g + \hat{u}_f + h\hat{\theta}} \quad (6)$$

where input is the total foundation displacement including rocking $u_g + u_f + h\theta$ and output is the total roof displacement $u_g + u_f + u + h\theta$.

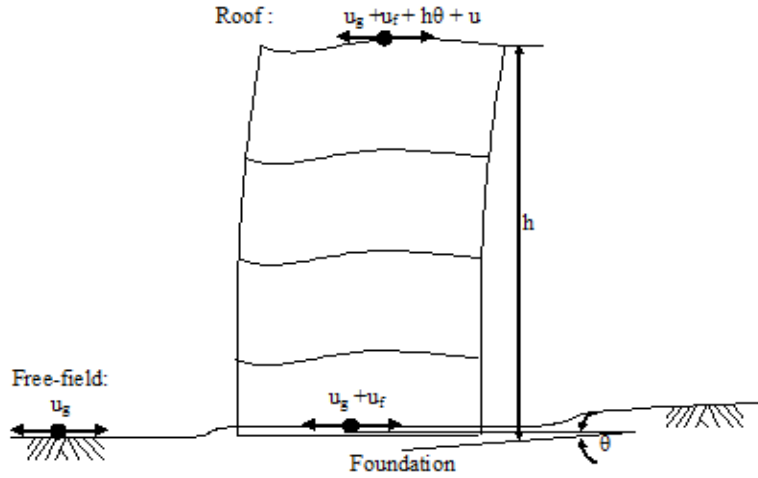


Figure 4. Inputs and outputs for evaluating SSI effects in system identification of buildings (Stewart & Fenves 1998)

The last two cases, i.e. pseudo flexible base and fixed base are relevant for this study because of available measurements. The Stewart and Fenves model for the fixed base summarized above is applicable to the case of a foundation where responses of all the degrees of freedom (DOFs) are available. In the analysed case of a 3D building on multiple footing foundation there are clearly more DOFs but the foundation response is measured only at four points for a total of 10 DOFs (see Figure 2). However, an assumption that the differential movement between the footings is very small and also lack of individual footing rocking (present in H_3) can be made which is valid for small level excitation. Thus, we argue that in our case pseudo flexible base system identification results will not be appreciably different from the fixed base results. However, for the sake of clarification and analysis, we have considered both pseudo flexible and fixed base models in this study to evaluate the differences, no matter how small these are. Stewart and Fenves (1998) demonstrate that the poles of the flexible base transfer function H_1 give natural frequencies and damping ratios of the entire dynamical system comprising the structure, foundation and soil. In other words, the identified modal

parameters are influenced by the translational and rotational stiffness and damping of soil. The natural frequencies and damping ratios identified from the poles of the fixed base transfer function H_3 , on the other hand, depend on the properties of the structure alone. The pseudo flexible base case is an intermediate one where the poles of transfer function H_2 yield modal parameters that depend on the stiffness and damping associated with the structure and foundation rocking (or more generally those foundation DOFs whose responses are ignored or not available). By comparing modal parameters identified from the different transfer functions the influence of the various types of foundation motions on those can be assessed.

To provide a simple quantification of the effects of SSI on the response of the building in this study, modal vibration parameters were sought through N4SID technique for the pseudo flexible base case and the fixed base case using input-output pairs consisting of a combination of foundation and superstructure level recordings as explained in Equations (5)-(6). For the pseudo flexible base case, horizontal translations of sensors 21, 22 and 23 were taken as the inputs while translational motions of sensors 25, 26, 27, 28, 29 and 2A as the outputs, whereas all three motions (two translations and one vertical) of sensors 21, 22, 23 and 24 as the input and all three motions of sensors 25, 26, 27, 28, 29 and 2A as the outputs for the fixed base case.

4 ANALYSIS OF THE SEISMIC RESPONSE OF THE BUILDING

The objective of the research is to assess and understand the seismic response of the building before and after the devastating earthquakes to ascertain the behaviour of the building system based on the frequency versus response levels. In particular, the trends are investigated between PRA of sensor 28 and the identified first three natural frequencies of the building using nine events. The presentation will thus follow selection of earthquakes, modal system identification and evaluating the difference in the behaviour of building before and after the strong events including effects of SSI.

4.1 *Earthquake records used in the analysis*

For this study, nine events are selected as shown in Table 1. Three of the events occurred in 2007, the next three are the Darfield main shock and two aftershocks, and the remaining three are the Christchurch main shock and aftershocks. The selection is intended to ascertain the response of the building before the first strong event had occurred, then during and after the first strong event, and lastly during the second strong event and after it. Based on the excitation and response levels before and after the strong events, the changes in the behaviour of the building is ascertained with fixed and pseudo flexible base models.

4.2 *Modal identification of the building*

N4SID technique was used to identify the first three frequencies, corresponding damping ratios and mode shapes. Sampling rate of the digitized signal was 200Hz and for establishing stabilization charts system orders from 2 to 150 were considered. A typical stabilization chart is shown in Figure 3: the marker 'black dot' shows all the identified frequencies, 'red dot' shows stable frequencies and damping ratios, while 'blue circle' stable frequencies, damping ratios and mode shapes. In this research, an identified frequency was considered to be stable if the absolute deviation between the frequency identified at the present and previous order was less than or equal to 0.01Hz. A stable damping ratio was defined by a deviation less than 5%. For mode shape stability, model assurance criterion (MAC) between the mode shapes of the present and previous order was to be at least 90% or greater. In this study, the first three modes were identified with confidence in all the events and subsequent discussions focus on these only.

For both pseudo flexible and fixed base models same mode shapes were observed in the planar view. The typical first three mode shapes of the building are shown in Figure 5. The levels of the building which had instrumentation are represented in the mode shapes in planar view. (Note that because of a limited number of measurement points those graphs assume the floors were rigid diaphragms.) The shape of the first mode shows it to be a translational mode along X-direction. The second mode is torsional and the third one is a translation along Y-direction.

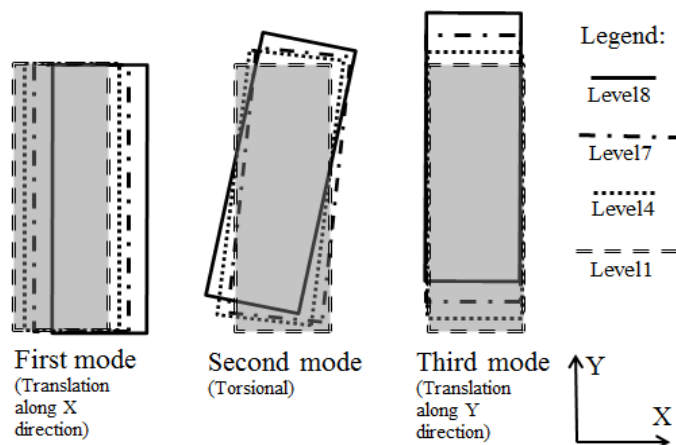


Figure 5. Planar views of the typical first three mode shapes

During some events, the second mode was missing in the system identification results (Table 1), which suggests that during those particular events this mode did not vibrate strongly enough. In an event, the second and third modal frequencies tended to be very close and the minimum difference between these two was found to be 0.04 Hz. This shows the capability of N4SID technique to identify very closely spaced modes.

4.3 Evaluating the difference in the behaviour of building considering SSI

Table 1 shows the identified frequencies and damping ratios for the analysed nine earthquakes for both fixed and pseudo flexible base models. The minimum, maximum and average values of the fixed base models are 1.65Hz, 2.68Hz and 2.20Hz for the first mode frequency, respectively, whereas they are 1.62Hz, 2.68Hz and 2.20Hz for the pseudo flexible base model. It can be observed that the fixed and pseudo flexible base model frequencies for the first mode are almost same except for the case of Darfield earthquake which shows 0.03Hz difference. However, it cannot be concluded that SSI is unimportant for the building first mode. As explained in the section 3.2, the difference between the fixed and pseudo flexible case is the inclusion of foundation rocking. However, due to the lack of a free field sensor the influence of foundation lateral motions could not be studied. It has been demonstrated that those effects can be significant (Butt & Omenzetter 2012). The pseudo flexible base case which according to the model represents the behaviour of the building including rocking effect (SSI) will be the focus of the discussion to assess the changes in the behaviour of the building for the selected earthquakes. There are, however, visible differences for the second and third modal frequencies. The maximum difference (0.31Hz) was found for the second frequency (torsional mode)

Table 1. Summary of identified frequencies and damping ratios for fixed and pseudo flexible base models

Earthquake	Fixed Base Model						Pseudo Flexible Base Model					
	Frequencies (Hz)			Damping Ratios (%)			Frequencies (Hz)			Damping Ratios (%)		
	1 st Mode	2 nd Mode	3 rd Mode	1 st Mode	2 nd Mode	3 rd Mode	1 st Mod	2 nd Mode	3 rd Mode	1 st Mode	2 nd Mode	3 rd Mode
03/05/2007	2.66	3.17	3.41	4.2	15.1	3.4	2.65	3.21	3.40	3.7	15.8	3.2
14/05/2007	2.64	-	3.34	3.6	-	5.0	2.64	-	3.33	3.8	-	4.7
17/05/2007	2.68	-	3.32	2.3	-	4.4	2.68	-	3.31	2.9	-	4.8
04/09/2010*	1.65	2.27	2.49	15.9	1.8	2.5	1.62	2.16	2.47	12.9	8.0	9.1
11/09/2010	2.18	2.79	3.09	3.3	4.7	8.2	2.18	-	3.00	2.2	-	6.3
06/11/2010	2.17	3.35	2.88	3.1	12.9	4.3	2.17	3.04	2.85	2.5	15.0	3.6
22/02/2011†	1.65	-	2.35	3.1	-	8.3	1.64	2.31	2.27	7.8	9.3	5.9
27/03/2011	2.13	-	2.93	4.6	-	7.5	2.12	-	2.89	5.0	-	8.3
30/03/2011	2.08	2.83	2.98	3.5	14.4	6.3	2.08	2.81	2.98	3.6	13.9	6.5
Minimum=	1.65	2.27	2.35	2.3	1.8	2.5	1.62	2.16	2.27	2.2	8.0	3.2
Maximum=	2.68	3.35	3.41	15.9	15.1	8.3	2.68	3.21	3.40	12.9	15.8	9.1
Average=	2.20	2.88	2.98	4.8	9.8	5.5	2.20	2.71	2.94	4.9	12.4	5.8
% Change‡=	47%	37%	36%	279%	135%	105%	48%	39%	38%	217%	63%	101%

*Darfield earthquake main shock; †Christchurch earthquake main shock; ‡% Change= (Maximum-Minimum)x100%/Average

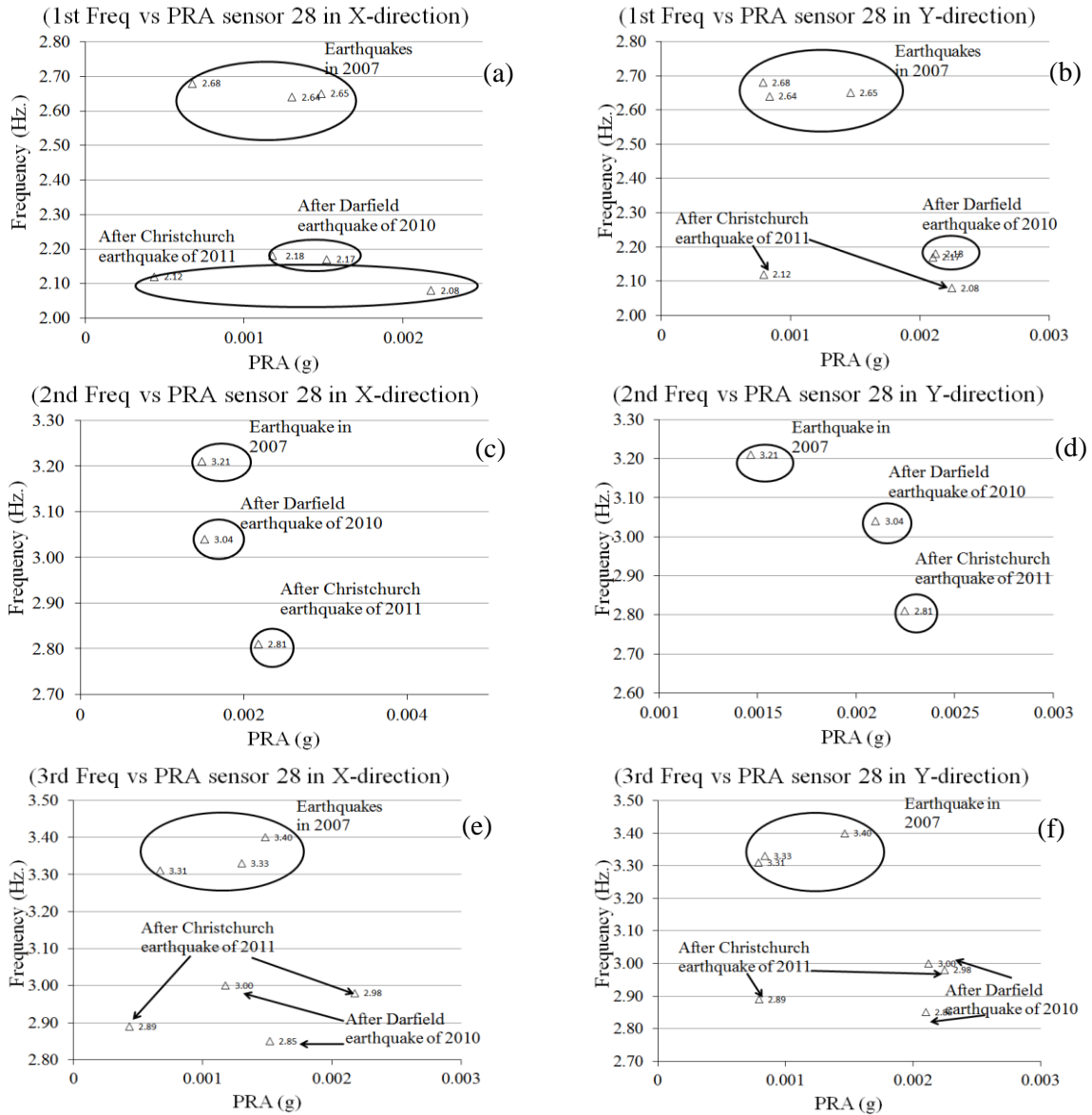


Figure 6. Frequencies versus PRA of sensor 28 for pseudo flexible base models along X (a, c and e) and Y directions (b, d and f).

during the 06/11/2010 event. Damping ratios, on the other hand, have large variation and the average values for the first, second and third modal frequencies for the fixed base model are 4.8%, 9.8% and 5.5%, respectively, whereas for the pseudo flexible base model these are 4.9%, 12.4% and 5.8%. Generally, the damping ratios for the pseudo flexible base model are higher than for the fixed base model except for a few cases (Table 1).

During the Darfield earthquake, the first mode frequency dropped to 1.62Hz, while in the Christchurch earthquake it was 1.64Hz for the pseudo flexible base models, which is a decrease by a factor of 1.63 (average frequency in 2007 events/frequency in the Darfield/Christchurch earthquake) compared to the pre-Darfield events. For the second mode frequency, the decrease is by a factor of 1.49 and 1.39, whereas for the third mode frequency it is 1.36 and 1.48 for the Darfield and Christchurch earthquakes, respectively. It has been shown by Celebi (2006) and Trifunac et al. (2001) that frequency decreases with the increase in excitation and response levels. To determine whether a similar behaviour is observed after the two main earthquakes, the first three frequencies of the pseudo flexible base model are plotted against PRA of sensor 28 (level 7) in both X and Y directions (Figures 6a-f). Sensor 28 was selected because for the 2007 events the sensors at level 8 were measuring only the vertical components so the response of the building cannot be compared with the pre-earthquake scenario by taking PRA of sensors at level 8, which would be the maximum response in the building.

It can be observed in Figure 6 (a, c and e) that after the Darfield earthquake for almost the same level of response for the pre and post-earthquake cases the frequencies are 18%, 5% and 30% less for the first, second and third modal frequencies, respectively, along X-direction ((average pre-earthquake frequency – average post-earthquake frequency)/average pre-earthquake frequency). After the Christchurch earthquake, for the response levels of 0.0005g and 0.002g the first mode frequencies are almost the same (2.12Hz, 2.08Hz) and correspond to a 21% decrease (Fig. 6a), while for the second mode frequency the decrease is 12.5%, and for third mode frequency for the response levels of 0.0004g and 0.002g the frequencies are very close (i.e. 2.89Hz and 2.98Hz). This shows a loss of stiffness along the X-direction after both the main events. The Y-direction plots (Fig. 6b, d and f) show very similar trends except for the second mode frequency plot (Fig. 6d).

It can be concluded from the above discussion that the building system (soil-foundation-structure) had experienced significant loss in stiffness after both the main events. However, this stiffness was partly restored after certain period of time, probably due to the soil healing capacity (Trifunac et al. 2001) by dynamic compaction caused by aftershocks and by settlement after certain time.

5 CONCLUSIONS

In this study, pre and post-earthquake seismic response analysis of an eight storey RC building was presented considering nine earthquake records. N4SID system identification technique was used to extract frequencies, damping ratios and mode shapes considering fixed and pseudo flexible base models. It was observed that the first mode frequencies have negligible differences, whereas the second and third mode frequencies have visible differences for the fixed and pseudo flexible base models. The behaviour of the building considering the pseudo flexible base model was ascertained after the Darfield (2010) and Christchurch (2011) earthquakes by plotting the identified frequencies vs. the PRA of a sensor at level 7. It was concluded that after both the main earthquakes the building system (soil-foundation-structure) has lost stiffness. This was, however, partly restored probably due to the healing capacity of soil.

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