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System identification and finite element modeling of a building incorporating soil flexibility and non-structural components

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ABSTRACT: In this study, system identification and finite element modeling of a four storey instrumented building is described. The reinforced concrete building has five tri-axial accelerometers to measure its response during seismic events. Subspace state-space system identification technique was used to extract dynamic properties including natural frequencies, damping ratios and mode shapes using recorded seismic responses. A three-dimensional finite element model of the building was developed to extract theoretical modal properties. To simulate real in-situ conditions, soil underneath the foundation and surrounding the basement was modeled using spring and dashpot elements and non-structural components (NSCs), such as cladding and partition walls, were also included. To evaluate the effect of soil and NSCs in dynamic response, a series of finite element models were constructed, viz. bare fixed-ended frame, frame with floor slabs, stairs, shear walls, NSCs and finally soil flexibility included. It was concluded from the investigation that the effect of soil and NSCs is significant towards the dynamic response of the building and these should be considered in models to simulate the real behavior.

1 INTRODUCTION

The full scale or in-situ experiments 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. 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. Celebi & Safak (1991) used Fourier spectra to identify the dynamic parameters from instrumented buildings under earthquake excitations. Along with the estimation of frequencies and damping ratios, they were also able to identify rocking of foundation showing soil-structure-interaction (SSI). Saito and Yokota (1996) used Autoregressive Moving Average model with Exogenous input to determine dynamic properties of an instrumented building during earthquake excitations. De Roeck et al. (2000) applied peak picking and stochastic subspace identification techniques to identify modal properties of a 15 storey reinforced concrete building with shear core under ambient vibrations. Stochastic subspace method was observed to be superior to peak picking as it identified frequencies which were missed by the peak picking method. Skolnik et al. (2006) performed system identification of the UCLA Factor Building, a 15 storey steel moment resisting frame, using low amplitude earthquake and ambient vibrations. State space subspace algorithm was used in the analyses and the measured responses were used to update finite element model of the building.

For performing response analyses, mathematical modeling of full-scale structures is required to produce models which can replicate true characteristics of the structures. An important factor in the modeling of civil engineering structures is the effect of soil-structure-interaction. Soil-structure-interaction involves transfer of energy from the ground to the structure and back to the ground (Trifunac & Todorovska 1999). Due to the flexibility of soil, the natural 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. Proper modeling of soil-structure-interaction is, therefore, necessary to better predict the actual response of structures. Bhattacharya & Dutta (2004) assessed the SSI effect by considering a number of scenarios in low-rise buildings on isolated grid footings and raft foundations using finite element modeling. The soil underneath the foundation was idealized using springs. Shakib & Fuladgar (2004) idealized soil as linear elastic solid elements and the contact surface between foundation and soil was modeled as linear plane interface elements with zero thickness. They deduced that SSI effects reduced the lateral and torsional displacements of asymmetric buildings causing a decrease in time period of the structure.

Another important aspect of structural modeling is the consideration of non-structural components (NSCs). It is a common practice to ignore NSCs like partition walls and claddings in a finite element model (FEM). But in many studies it has been demonstrated that while the effect of a single NSC on dynamic response of a building can be negligible their cumulative effect can be significant (Su et al. 2005). The numerical results achieved by modeling of the NSCs reveal the participation of these components during shaking of the structures. The level of participation depends upon the extent of shaking. Usually the plasterboard clad walls are considered to be providing no significant contribution towards lateral stiffness. But it was shown by physical testing by Liew et al. (2002) that these types of walls provide lateral stiffness and strength during seismic events. It is therefore necessary to incorporate the effect of NSCs adequately into FEM.

This study comprises two parts. In the first part, dynamic properties of the instrumented building will be extracted from its earthquake responses using the subspace state-space identification technique. For natural input modal analysis, this technique is considered to be the most powerful class of the known system identification techniques in the time domain (Van Overschee & De Moor 1994). The second part is a study of the building dynamics by incorporating the soil flexibility and NSCs. To ascertain the influence of structural and non-structural components, FEMs were constructed considering different cases, i.e., bare fixed base frame, frame with slabs, lift shaft, NSCs, soil under foundation and building partly submerged in soil, respectively. The FEM and measured results are then compared. The study attempts to highlight the importance of modeling the soil and NSCs to simulate the real behavior of the structures and is expected to further the understanding of the dynamic response of buildings.

2 SYSTEM IDENTIFICATION USING EARTHQUAKE EXCITATION

2.1 Description of the building and sensor array

The building under study is a four storey reinforced concrete structure with a basement, located in Lower Hutt approx. 20km north-east of Wellington, New Zealand. The structural system con-

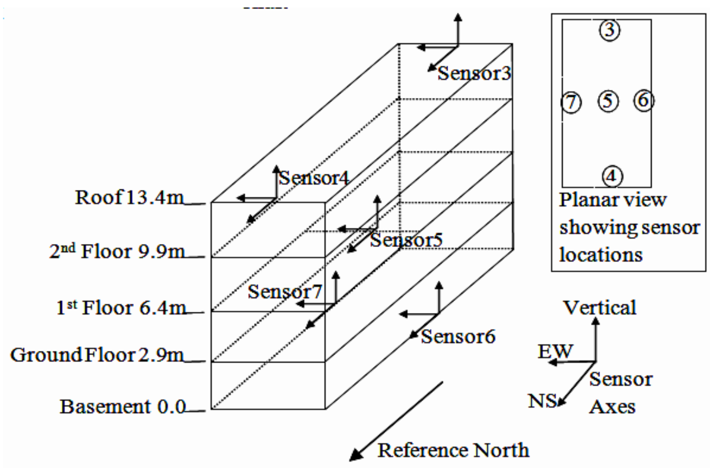

Figure 1. Three dimensional sketch of the building showing sensor array marked with sensor numbers and their sensitive axes. Inset shows a planar view marked with sensor locations.

sists of beam-column frames with a reinforced concrete shear core (wall thickness 229 mm) which houses an elevator. The frame arrangement is regular and symmetrical in plan but the elevator shaft near the north end makes the whole building unsymmetrical in terms of stiffness distribution. All the exterior beams are 762 × 356 mm except at the roof level where these are 1067 × 356 mm. All the interior beams and columns are 610 × 610 mm. Floors are 127 mm thick reinforced concrete slabs except a small portion of the ground floor near the stairs where it is 203 mm thick. The roof comprises corrugated steel sheets over timber planks supported by steel trusses. The columns are supported on separate pad type footings of base dimensions 2.29 m x 2.29 m (at the perimeter) and 2.74m x 2.74m (inside the perimeter) and tie beams of 610 × 356 mm are provided to join all the footings together. In the basement, retaining walls, not connected to the columns, are provided at all the four sides. The building has been instrumented as part of the GeoNet project (www.geonet.org.nz), with five tri-axial accelerometers. Two accelerometers are fixed at the base level, two at the roof level and one underneath the first floor slab as shown in Figure 1. All the data are stored to a central recording unit and are available online (www.geonet.org.nz).

2.2 *Subspace state-space identification technique*

The subspace state-space identification technique (Van Overschee & De Moor 1996) derives state-space models for linear systems by applying the well-conditioned operations, like singular value decomposition, to the block Hankel data matrices. After sampling of the continuous time state space model, the discrete time state space model can be written as:

$$\boldsymbol{x}_{k+1} = \boldsymbol{A}\boldsymbol{x}_k + \boldsymbol{B}\boldsymbol{u}_k \tag{1}$$

$$\boldsymbol{y}_k = \boldsymbol{C}\boldsymbol{x}_k + \boldsymbol{D}\boldsymbol{u}_k \tag{2}$$

where $\boldsymbol{A}$, $\boldsymbol{B}$, $\boldsymbol{C}$ and $\boldsymbol{D}$ are the discrete state, input, output and control matrices respectively, whereas $\boldsymbol{u}_k$ is the excitation vector and $\boldsymbol{x}_k$, $\boldsymbol{y}_k$ are discrete time state and output vectors, respectively. In reality, there are always process and measurement noises present so adding these to the above equations results in:

$$\boldsymbol{x}_{k+1} = \boldsymbol{A}\boldsymbol{x}_k + \boldsymbol{B}\boldsymbol{u}_k + \boldsymbol{w}_k \tag{3}$$

$$\boldsymbol{y}_k = \boldsymbol{C}\boldsymbol{x}_k + \boldsymbol{D}\boldsymbol{u}_k + \boldsymbol{v}_k \tag{4}$$

Here $\boldsymbol{w}_k$ and $\boldsymbol{v}_k$ are the process and measurement noises, respectively. The identification involves two steps. The first step takes projections of certain subspaces calculated from input and output observations 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 $\boldsymbol{A}$, $\boldsymbol{B}$, $\boldsymbol{C}$ and $\boldsymbol{D}$. Then the modal parameters, i.e. frequencies, damping ratios and mode shapes, are found by eigenvalue decomposition of the system matrix $\boldsymbol{A}$.

In order to determine the proper system order, the trend of the estimated modal parameters in a stabilization chart is observed as the system order increases sequentially. Stability tolerances are chosen based on the variance in frequency, damping ratios and mode shapes among the considered system orders.

2.3 Identification results

For this study, the earthquake of 18th November 2009, which had epicenter at 10 km south of Palmerston North, was selected. This earthquake has a Richter magnitude of 5.1 and peak ground acceleration (PGA) of 0.002g as measured at the base of the building and peak response acceleration (PRA) of 0.015g at the roof as shown in Figures 2 and 3 respectively. The reason for adopting this earthquake was to select an event of an intensity capable of strongly exciting all the modes of interest. The building is located in an area which has not been hit by a strong earthquake since its instrumentation and the selected earthquake is one of the very few moderate intensity events.

For the system identification, sensors 6 and 7 were taken as the inputs (excitations) while sensors 3, 4 and 5 as the outputs (responses) (see Figure 1). Sampling rate was 200 Hz and for establishing the stabilization diagram (Figure 4), system orders from 60 to 160 were evaluated. Stability tolerances are chosen based on the relative change in the modal properties of a given mode as the system order increases. An identified frequency was said to be stable if the absolute deviation between 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 shapes stability, model assurance criterion (MAC) between the mode shapes of the present and previous orders was to be at least 95% or greater. MAC is actually an index to determine the similarity between the two mode shapes. For modes $\boldsymbol{\phi}_i$ and $\boldsymbol{\phi}_j$ the MAC is defined as:

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

In equation superscript T shows the transpose of the matrix. In the Figure 4, the marker sign “dot” shows all the identified frequencies, the “plus” sign the stable frequencies with damping ratios, while “plus with circle” sign the stable frequencies with damping ratios and mode shapes. The identified first three frequencies are 3.07 Hz, 3.45 Hz and 3.64 Hz and the corresponding damping ratios are 4%, 3.4% and 4% respectively. The identified mode shapes are shown in Figure 5 in planar view. The shape of the first mode shows it to be a translational mode along the east-west (EW) direction with some rotation. The second mode is nearly purely torsional and the third one is a translation dominant mode along the north-south (NS) direction coupled with torsion. The shear core present near the north side creates an unsymmetrical distribution of stiffness and is the primary cause of the torsional behavior in all of the three modes.

3 FINITE ELEMENT MODELING

To analyze the dynamic behavior of the building considering the soil flexibility and NSCs, a three dimensional FEM was developed using provided structural drawings and additional at-site measurements. The beams and columns were modeled as two node line elements, and slabs,

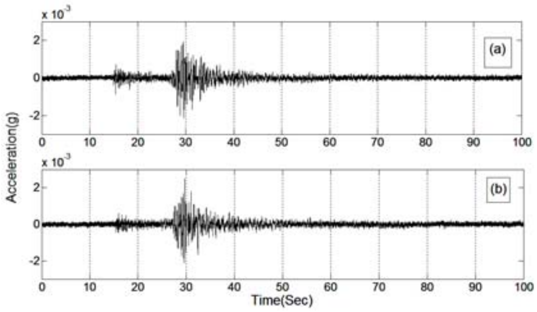

Figure 2. Building base seismic acceleration time histories from sensor 6: (a) EW-component (b) NS-component (PGA).

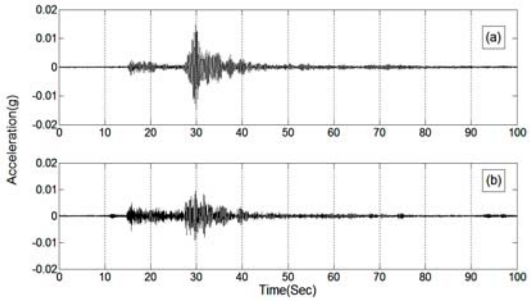

Figure 3. Building roof seismic acceleration time histories from sensor 4: (a) EW-component (PRA) (b) NS-component.

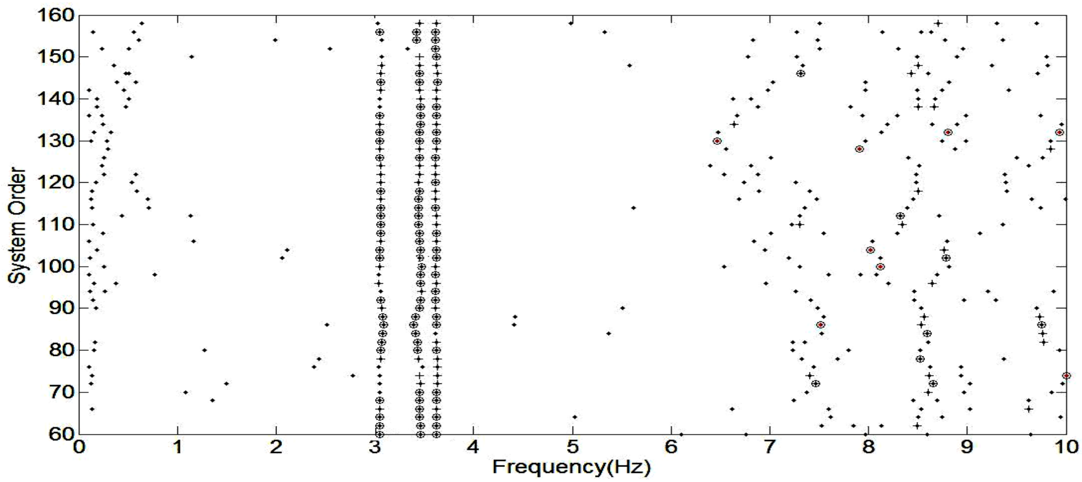

Figure 4. Stabilization diagram showing the trends of frequencies. Marker sign "dot" shows all the identified frequencies, "plus" sign the stable frequency and damping ratio, while "plus with circle" sign the stable frequency with damping ratio and mode shapes.

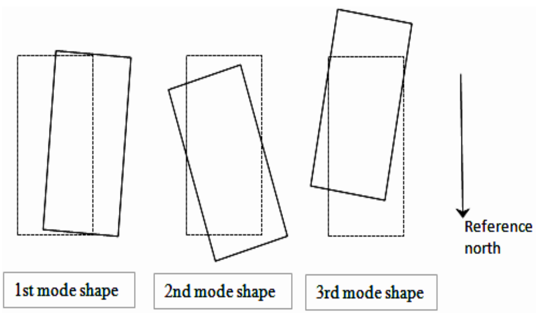

Figure 5. Planar views of the first three mode shapes identified using subspace state-space identification method.

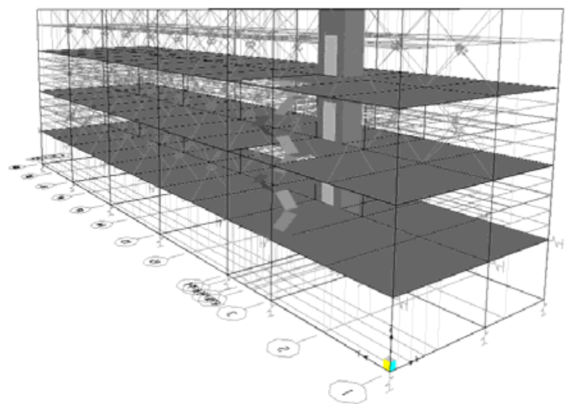

Figure 6. Three dimensional FEM showing shear core, soil springs, partition walls and stairs. Cladding has been removed from the view to show the inner details.

stairs and shear core as four node shell elements. Linear elastic material properties were considered for the analysis. Initially the base was assumed as fixed and beam to column connections were also assumed as fixed (moment resisting frame assumption). The density and modulus of elasticity of concrete for all the elements was taken as 2400 kg/m^3 and 25 GPa except for the shear core for which it was 2550 kg/m^3 and 26.5 GPa, respectively. The steel density and modulus of elasticity for roof elements were taken as 7800 kg/m^3 and 200 GPa, respectively. The trusses present at the roof level were modeled as equivalent steel beams. The masses of the timber purlins, planks and steel corrugated sheet were calculated and lumped at the beams. All the dead and superimposed loads were applied as area loads or line loads at their respective positions. Figure 6 shows the three dimensional FEM having structural elements and NSCs (cladding, partition walls) and soil flexibility modeled in it.

Initially, the bare frame model with fixed base was developed which includes only the beams and columns. Then a series of FEMs were developed to ascertain the influence of the different structural elements and NSCs namely:

(a) bare fixed base frame;
(b) frame with slabs;
(c) frame with slabs and lift shaft;
(d) frame with slabs, lift shaft and NSCs modeled;
(e) frame with slabs, lift shaft, NSCs and soil underneath foundation modeled;
(f) frame as in (e) with modeling of the building partly submerged in soil.

3.1 Idealization of soil

Soil present at the site was classified according to the New Zealand Standard NZS1170 as class D (deep or soft soil). No other information was available regarding the type of soil. It was assumed that it is clay and corresponding properties for shear modulus, Poisson's ratio, density and shear wave velocity were taken from literature (Bowles 1996).

In FEM soil was idealized using springs and dashpot elements. The values of static stiffness and radiation dashpot coefficients were calculated using the equations provided in Gazetas (1991). The dynamic stiffness coefficients were calculated by looking up the values of a_o ($=\omega B/V_s$) and L/B from the charts provided in the above reference. Here ω is the circular frequency equal to $2\pi f$ and this is one of the earthquake dominant frequencies in the case of seismic excitation, whereas B, L and V_s are the half width, half length of the base of footing and the shear wave velocity respectively. The total stiffness was calculated by multiplying the static stiffness by dynamic stiffness coefficients. Material dashpot constant $2K\beta/\omega$ was added to the radiation damping coefficient to estimate the total damping coefficient. Here K is the total stiffness calculated above and β is the soil or material damping ratio which was taken as 0.05. A range for β between 0 and 0.10 is suggested by Whitman & Richart (1967). Since the building is partially submerged in soil, equivalent horizontal soil springs and dashpots were calculated using the submerged column as a footing and were included in the FEM in the last stage (f).

3.2 Idealization of NSCs

External cladding in the building is made up of fiberglass panels with insulating material on the inner side. The density and modulus of elasticity values of fiberglass were taken from literature (Gaylord 1974) as 1750 kg/m^3 and 19 GPa, respectively. Claddings were modeled as four node shell thin elements. Since the structure under study is an office building, there are a large number of partition walls present. The stiffness values of gypsum wall partitions were taken from Kanvinde & Deierlein (2006) as 2800 kN/m. The mass of the partition walls were calculated manually and applied as area mass of 20 kg/m^2. Gypsum partition walls were modeled as two joint link elements which are diagonal elements representing only stiffness along the longitudinal direction. The mass of false ceiling was taken as area mass of 12 kg/m^2.

3.3 Results of FEM eigenvalue analysis and discussion

The results of FEM eigenvalue analysis are presented in Table 1. An important observation from the analysis is that the values of frequencies of bare frame are much lower compared to the subsequent models. Stage (b) adds slabs in the bare frame increasing the stiffness and mass. Stage (c) includes shear core in the model which has increased the EW, torsional and NS mode frequencies by 2%, 11% and 12% respectively from the previous stage. The values also depict the influence of NSCs at stage (d). The frequencies have changed significantly from the previous stage by 22%, 35% and 38% for EW, torsion and NS directions respectively. By modeling the soil in stage (e) the frequencies are reduced by 21%, 26% and 19% respectively from the previous stage for EW, torsion and NS directions, which shows substantial influence of soil on the dynamic properties. The final stage (f) includes modeling of the partial submersion of building in which the torsional and NS mode frequencies are well in agreement with experimental values but the EW frequency has still 18% difference with the measured value, although it has much improved from the previous stages. EW (lateral) direction is shorter in length and less stiff as compared to the NS (longitudinal) direction. One of the reasons of this frequency difference can be the assumed soil properties being different from the actual in-situ properties. Also the soil surrounding the building along the EW and NS direction can be different from the soil underneath the foundations. It therefore would be imperative to confirm the actual properties of soil and then use these for subsequent analyses. For this purpose an experiment using forced vibrations imparted by a shaker and Spectral Analysis of Surface Waves (SASW) method will be used to determine the in-situ characteristics of soil at the site. This study can be considered as an initial investigation to see the influence of soil on dynamic characteristics. Soil properties therefore were assumed for this analysis for the time being. Model updating would be the next step after incorporating the in-situ soil characteristics in FEM to remove the differences between system identification and FEM results.

Table 1. Comparison of modal analysis results of various stages of FEM with identification results

	Modal frequencies (Hz)						
Modes	(a)	(b)	(c)	(d)	(e)	(f)	Measured value
EW Trans.	2.1 (32%)	1.87 (39%)	1.94 (37%)	2.61 (15%)	1.95 (36%)	2.53 (18%)	3.07
Torsion	2.29 (34%)	2.09 (39%)	2.47 (28%)	3.69 (-7%)	2.8 (19%)	3.42 (1%)	3.45
NS Trans.	2.31 (37%)	1.94 (47%)	2.38 (35%)	3.76 (-3%)	3.06 (16%)	3.65 (0%)	3.64

Note: The values in parenthesis show the percentage difference of the particular FEM stage results and experimental results.

4 CONCLUSIONS AND FUTURE RESEARCH

The objective of this research has been to highlight the importance of soil and NSCs modeling in the FEM of buildings. Firstly, system identification of the building was carried out under an earthquake excitation and then FEMs were developed in stages to see the influence of different elements of the structure. The following important conclusions are drawn from the study:

1. NSCs significantly increase the frequencies by 22%, 35% and 38% for EW, torsional and NS directions, respectively, while including soil properties in the model reduced the above modal frequencies by 21%, 26% and 19%, respectively.
2. Modeling the partial submersion in soil improved the agreement between experimental and FEM frequencies appreciably, except for the EW direction.
3. To simulate the true soil behavior, actual in-situ soil characteristics will be incorporated in the FEM by testing the soil at site and using the SASW method.
4. Model updating will be carried out to improve the agreement between FEM and measured dynamic characteristics.

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