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VALIDATION OF A NUMERICAL MODEL FOR SEISMIC ANALYSIS OF A TELECOMMUNICATION TOWER MOUNTED ON A BUILDING ROOFTOP IN TAIWAN

Rola Assi[1], Sébastien Langlois[1], Ghyslaine McClure[1], C.Y. Chen[2], George C. Yao[2]
1. Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada
2. Department of Architecture, National Cheng Kung University, Tainan City, Taiwan (ROC)

ABSTRACT: This paper discusses the validation of two models of a five-floor concrete building located in Taiwan. The building supports a 10-m high self-supporting lattice telecommunication tower on its rooftop. The purpose of modelling is to assess the functionality of the tower and operational and functional components attached to the building during and after earthquakes. Monitoring data available includes 18 floor acceleration records at various levels from the basement to the roof, and three acceleration records at the tower mid-height. The combined building-tower structure was modelled using the commercial software SAP2000. Dynamic time history analyses were performed using basement acceleration records from the 1999 Chi Chi earthquake. The calculated floor and tower accelerations are compared to the recorded data. Of particular interest are the amplification of the floor accelerations along the building height, from the basement to the roof, and the dynamic amplification of the tower response with respect to the response at roof level.

1 INTRODUCTION

Dynamic analysis of structures is an important step in the process of developing design standards and codes for structures such as buildings and towers subjected to strong earthquakes. This allows to improve provisions and recommendations for design of operational and functional components (OFCs) and lifelines attached to these structures as well.

In fact, the development of seismic design provisions for non-structural components has lagged behind that of primary structures. The seismic vulnerabilities of OFCs in modern buildings were not exposed until the 1964 Alaska and 1971 San Fernando earthquakes (Lagorio 1990, Reitherman 1997). More recently, there has been an increasing concern about the seismic performance of OFCs attached to primary structures. A review of the typical damage sustained in recent earthquakes (Soong 1990; McKevitt et al. 1995; Phipps 1997; Kao et al. 1999; Naeim 1999,2000; Filiatrault et al. 2001) highlights the fact that the unsatisfactory performance of non-structural components, equipment and systems is the greatest contributor to damage, losses and business interruption in most facilities.

A few countries in high seismic activity zones have extensive instrumentation programs. Structures are equipped with accelerometers to measure the intensity of the earthquakes as well as the response of the structures to shaking, specifically accelerations. The accurate prediction of accelerations is important for the design of OFCs, especially at roof levels where the accelerations are amplified up to three times according to the currently suggested provisions (NRCC 2005). The calibrated models of existing buildings using commercial softwares allow studying different variables on the amplification of accelerations.

Current seismic provisions in codes and standards for telecommunication towers relate to structures on ground and are not specific for towers on rooftops. The design of these towers is typically controlled by extreme wind, ice and wind combinations, and restrictive deflection limits (TIA/EIA 222-G 2002, CSA-S37 2001), and hence most codes and standards are concerned with wind and ice loads in cold regions. However when the tower supports heavy attachments at the upper part or in the case of uneven distribution of rigidity and/or mass, or when the tower is erected on a building, it becomes necessary to check its seismic response (its base shear in particular) in an area prone to earthquakes.

In this paper, experimental recorded accelerations of a building-tower structure are compared to values obtained from a three-dimensional model of the structure using SAP 2000. This work allowed to verify the accuracy of the modelling approach generally used to represent a building or a tower with commercial structural analysis software. The calibrated numerical model will allow to study the important parameters that influence the amplification of accelerations along the building elevation.

2 ANALYSIS

2.1 DESCRIPTION OF THE BUILDING-TOWER STRUCTURE

Taiwan is located in a highly active seismic area. This island is subjected to strong earthquakes several times every decade and moderate intensity earthquakes are very frequent. Many structures in cities have been equipped with accelerographs, which were installed under the Taiwan Strong Motion Instrument Program (TSMIP) between 1992 and 1997, a network that covers the entire island (Shin 2000). The structure studied is one of these instrumented buildings.

The structure is a five-floor concrete shear-wall building owned by China Telecom and located in a small town of Tainan County. It has an irregular geometry and the configuration of shear walls changes from one storey to another. Of particular interest is that the building was extensively instrumented at several locations and that it supports on its rooftop a 10m, square base 4.7m$\times$4.7m self-supporting steel lattice telecommunication tower. The tower was also equipped with sensors in the x, y and z directions at its mid-height. Elevation views of the instrumented building and tower are shown in figures 1, 2 and 3. Available records are those of the 1999 Chi Chi earthquake of magnitude 7.6, the largest inland earthquake in Taiwan in the 20th century.

Figure 1. China Telecom Building in Tainan County (Taiwan)

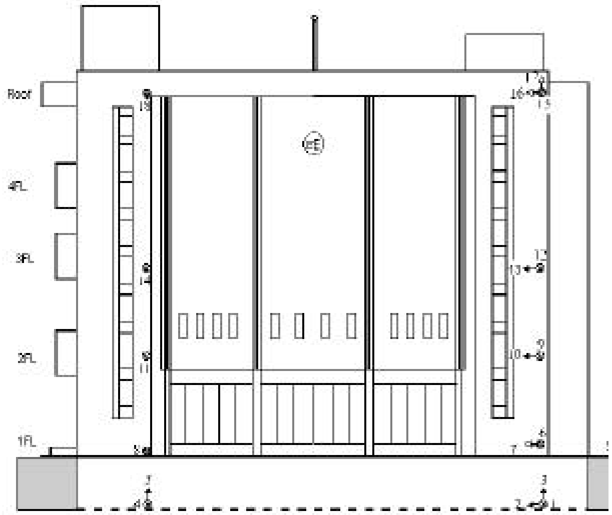

Figure 2. Building elevation

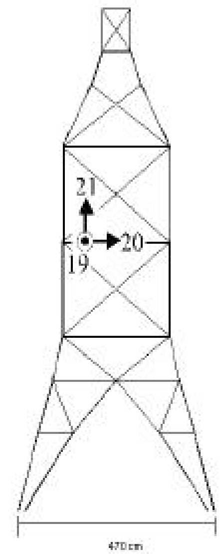

Figure 3. Tower elevation
(Simplified model)

2.2 MODELLING

The structure was modelled using SAP 2000. The tower was modelled as a frame-truss and attached to the building by four rigid members protruding from the rooftop (see figures 4 and 5).

Figure 4. Tower base

Figure 5. Close-up of tower support

Rotational degrees of freedom were released from secondary members including all diagonal members and horizontal members. Their equivalent mass was manually lumped to joints located on the main legs in the x, y, and z directions. Additional masses of the connection plates, secondary members, railing and antennas were also considered.

Two three-dimensional models were created for the building. In the first model, an approximate position of the centre of mass was calculated and shear walls of equivalent properties at predefined positioned (see figure 6) were modelled. Only the columns and shear walls were modelled and a displacement constraint was used to simulate a rigid diaphragm effect. The masses of floors, beams, columns and shear walls were calculated and assigned to the centre of mass in the x and y directions (horizontal). In the vertical direction, the mass was distributed according to tributary area of columns and shear walls. The in-plane rotational inertia of the slab was also considered. The building was assumed perfectly fixed at the base.

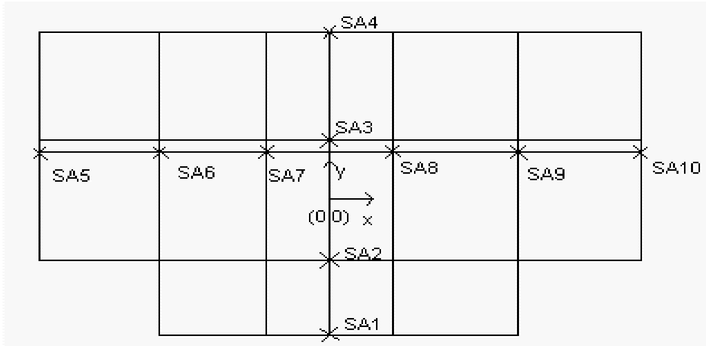

Figure 6. Location of equivalent shear walls for model 1

The second model was constructed in the same way as the previous one except that the shear walls were modelled at their exact position and the floor beams were also included. Masses of diaphragm, external and inner walls were lumped in the x, y and z directions at nodes according to tributary area of walls and columns. The basement and the first floor peripheral joints were fixed.

In both models, the recorded accelerograms at the basement were used as input. Elastic time-history modal superposition analysis was used, and thirty modes were considered.

The objective of creating two models was to study whether the simplifying assumptions of the first model would affect the accuracy of the results. The calibrated numerical models will allow to study the effect of important parameters that influence the amplification of accelerations such as the input ground motion, rigidity, mass, height of the building.

3 RESULTS

3.1 HORIZONTAL ACCELERATIONS

The linear distribution of accelerations along the building height constitutes a significant change in the NBCC provisions (Assi 2003). The height factor was introduced for the first time in the 1995 edition of the NBCC (NRCC 1995), for mechanical and electrical components, and for all components (mechanical, electrical and architectural) in the proposed 2005 edition (NRCC 2005). In order to get a better understanding of the amplification of accelerations along the height, two analyses were carried out on the structure: time-history elastic analysis and analysis of data from the instrumented building. The natural periods of the building were also estimated by using the transfer function of the roof and basement channels. Figures 7 and 8 illustrate the variation of accelerations along the height of the building.

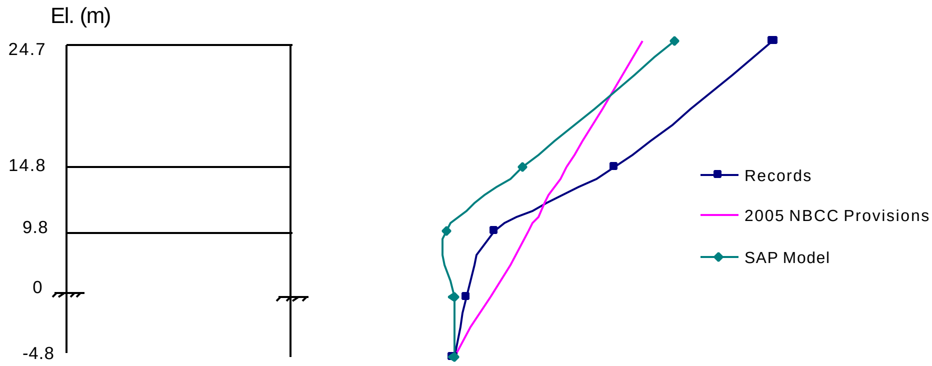

Figure 7. Horizontal acceleration profiles in the case study building

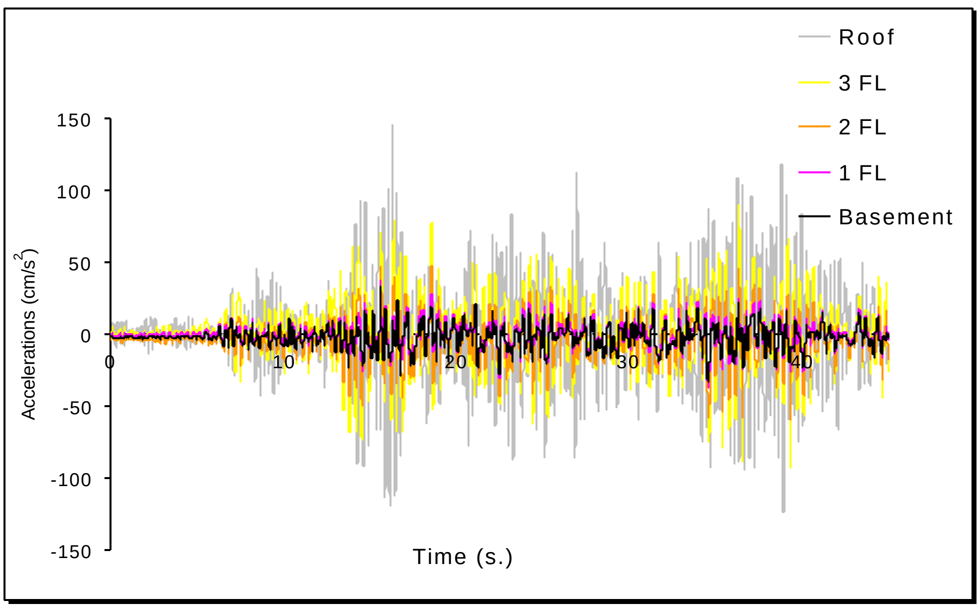

Figure 8. Amplification of accelerations from the basement to the rooftop

3.2 OBSERVATIONS FROM RECORDED ACCELEROGRAMS

The Fast Fourier Transforms (FFT) of the recorded accelerograms were generated to identify the natural periods of vibration of the building. Figure 9 shows the FFT of channels 1, 6, 9, 12 and 15 in the y-direction. The peaks correspond to the predominant frequencies of the structure and the input ground motion. It is clearly seen that the frequency content of the input motion was amplified and modified along the building height.

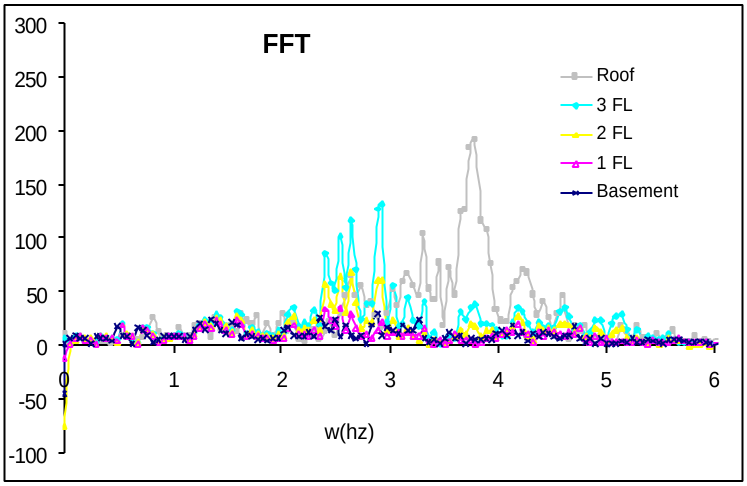

Figure 9. FFT of records in the y-direction

Figure 10 illustrates the variation in the response between the rooftop and the mid-height of the tower (channels 15 and 19). Again, the amplification of the horizontal accelerations is clear. Also the graph shows the differences in the frequency content of the response of the roof and the tower, confirming the flexibility of the tower relative to that of the building.

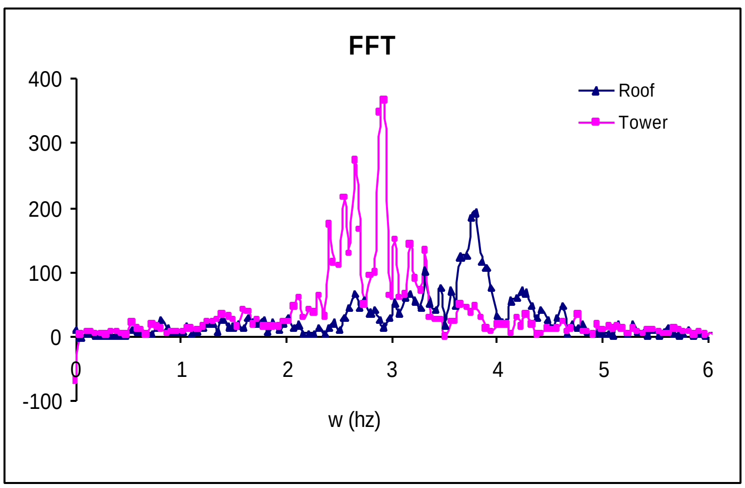

Figure 10. FFT of records in the y-direction

3.3 RESULTS FROM DETAILED SAP MODELS

The SAP models allow to study important parameters affecting the seismic response of both the building and the tower. Available records served to calibrate the model. The FFT of accelerations obtained from model 2 for channels 1, 6, 9, 12 and 15 are illustrated in figure 11.

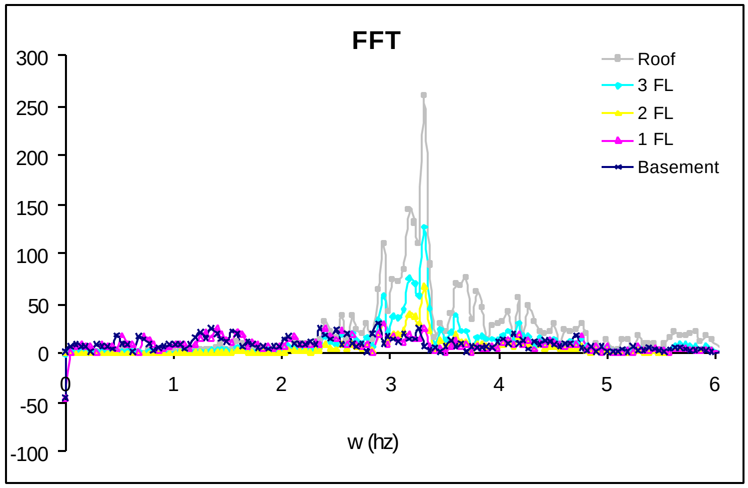

Figure 11. FFT of accelerations from the SAP model

Accurate prediction of horizontal accelerations at the roof level (ch15) is of particular interest because it can serve as input for the seismic design of the tower. Calibrated model structures allow to study the effect of different parameters such as the type of lateral load resisting system, the building height and the input ground motion on the amount of amplification of accelerations. Figure 12 shows the correlation between the records and the two SAP models. The period of vibration of the building alone was calculated and compared to the building-tower period; it was found the influence of the tower is negligible on modes of the building. Vertical (axial) modes occur at high frequencies (above 7 hz), as expected, because the building is very stiff in the vertical direction.

Table 1 summarises the results obtained for the first three natural modes of vibration computed using the records, the SAP models and the building alone. An ambient vibration analysis was also performed in Taiwan and these results are added in Table 1.

Table 1: Summary of identification of natural frequencies of the structure

Building direction	Ambient Analysis	Chi-Chi EQ Analysis	SAP2000 (Building-tower Model 2)	SAP2000 (Building alone)
Y-translation	3.52 Hz	3.54 Hz	3.34 Hz (flexural)	3.36 Hz
X-translation	4.35 Hz	3.87 Hz	3.81 Hz (flexural)	3.87 Hz
Torsion			5.76 Hz (torsional)	5.76 Hz

3.4 COMPARISON OF SAP MODELS 1 AND 2

By comparing the results of the two SAP models with the Chi Chi earthquake records for channel 15 (figure 12), it is noticed that the peaks of both models occur at the same frequency. Model 1 is more simplified than model 2 however it gives very good results. The first natural periods of vibration of the structure calculated for models 1 and 2 are equal to 0.29 s and 0.3 s respectively, so the difference for model 1 relative to model 2 is 4% only. Both numerical models are more flexible than the existing structure; this is due in part to modelling assumptions and approximations in the calculation of the mass and the stiffness of the structure. The calculated period from the Chi Chi records is equal to 0.28 s.

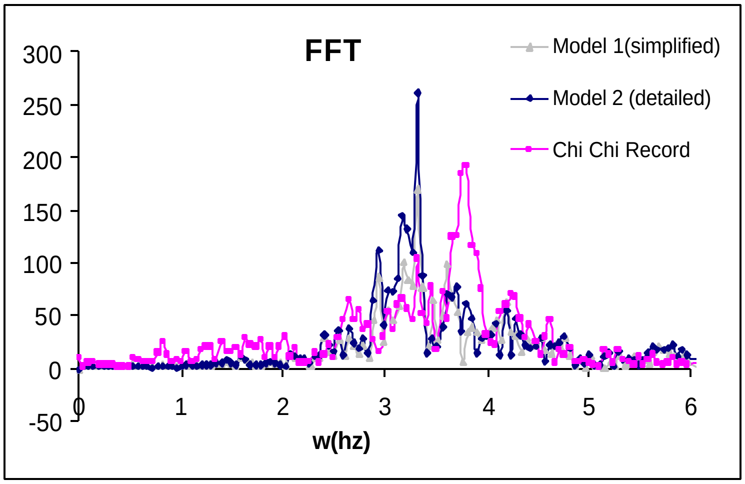

Figure 12. FFT of rooftop accelerations in the y-direction

4 CONCLUSIONS

Two 3-D models were created for a building supporting a steel lattice telecommunication tower on its rooftop. Accelerations from both models were compared to recorded accelerograms from the 1999 Chi Chi earthquake for the sake of studying the effect of simplified modelling assumptions on the accuracy of results. It was found that the simplifications in the first model did not affect the accuracy of the results. The calibrated models will later serve to study the relative importance of different parameters on the variation of floor accelerations along the building elevation.

This case study is part of a research program that aims to improve the prediction of accelerations for common buildings, especially at the rooftop. An accurate prediction of acceleration on rooftop of buildings is very useful in developing simplified methods for lattice telecommunication towers mounted on rooftops and other heavy OFCs.

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