13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1988

SEISMIC CONSIDERATIONS FOR TELECOMMUNICATION TOWERS MOUNTED ON BUILDING ROOFTOPS

Ghyslaine McCLURE[1], Laura GEORGI[2] and Rola ASSI[3]

SUMMARY

Lattice towers are used as antenna-supporting structures in telecommunication network systems. Some of these telecommunication towers represent critical links in a network and may be required to be fully functional during or immediately after a strong earthquake. In a densely built environment, towers are mounted on building rooftops and their response is influenced by the dynamic characteristics of the building itself.

In this study, time history analyses are used to explore the correlation between the building accelerations and the maximum seismic base shear as well as the base overturning moment of towers mounted on building rooftops. The models include two medium-rise buildings combined with two self-supporting lattice steel towers subjected to 45 horizontal accelerograms with varied frequency content. The tower base shear results are compared with the predictions based on a simplified formula proposed in building codes for secondary structures. Although data for the isolated buildings and towers is realistic, the combined structures do not exist.

INTRODUCTION

Self-supporting telecommunication towers are three-legged or four-legged space trussed structures with usual heights between 30 m and 160 m. They consist of main legs and horizontal and vertical bracing members. Main legs are typically composed of 90° angles in four-legged towers and 60° angles in three-legged towers. The most common brace patterns are the chevron and the X-bracing. Commonly, lattice towers are built on firm ground, but to avoid large and expensive constructions as well as due to space constraints in urban areas, they are sometimes mounted on building rooftops. Hence, these towers are usually shorter than those built on ground.

Generally, the design of self-supporting towers takes into account the effect of wind and ice loads as the only source of environmental loads. Except for critical structures built in high seismic hazard areas, earthquake-induced loads are generally neglected in design. This practice is mainly due to the fact that

[1] Associate Professor, Civil Engineering and Applied Mechanics, McGill University, Canada
[2] Visiting Researcher (2002), Graduate Student at Chalmers University of Technology, Sweden
[3] Ph.D. Candidate, Civil Engineering and Applied Mechanics, McGill University, Canada

lattice towers built on ground have shown good performance in past earthquakes. However, towers mounted on rooftops respond to seismic motions differently than those built on firm ground. It can be time consuming to perform a detailed dynamic analysis as part of a regular design project and such an analysis is not always necessary. Thus, simplified design checks need to be developed.

Research on the seismic response of lattice towers has been performed at McGill University since 1992. The goal of these studies was to find simple predictors and procedures that can be used by tower designers to determine whether a given tower will be sensitive to seismic effects and whether detailed dynamic analysis is necessary. However, the research on towers mounted on building rooftops is still in progress.

This work uses numerical simulations to study the seismic response of two self-supporting telecommunication lattice towers of heights of 30 m and 40 m, mounted on the rooftop of two medium-rise buildings: Burnside Hall, which is located on McGill Campus, and 2020 University, which is located nearby in downtown Montreal. The goal is to explore the correlation between the earthquake-induced motion of the building and the tower base shear force as well as the tower base overturning moment.

SEISMIC SENSITIVITY OF SELF-SUPPORTING TELECOMMUNICATION TOWERS

Besides the dynamic characteristics of the towers and the specific signature of the input ground motion, one can identify three main factors governing the design of towers for earthquake effects: the seismicity of the tower location (hazard level), the tower serviceability requirements (related to functionality), and geotechnical considerations. In case of towers on building rooftops, the geotechnical considerations are further compounded with the dynamic support conditions provided by the building itself.

For Canada, information about the seismicity of the site can be found in uniform seismic hazard maps and corresponding earthquake data is prescribed by the National Building Code of Canada (NBCC) (NRC/IRC, proposed 2005). These hazard levels were particularly selected for buildings, to ensure life safety by resisting moderate earthquakes without significant damage, and major earthquakes without collapse. However, life safety is not the only performance objective appropriate for the design of telecommunication towers. Depending on the economical and/or strategic value of the function of the tower and the consequences of its failure or lack of serviceability, the tower owner should choose one of the following three performance levels:

- Life safety: No collapse of the tower in a life-threatening way.
- Interrupted serviceability: Serviceability of the tower after, but not during the earthquake.
- Continuous serviceability: Full serviceability of the tower during and after the earthquake.

According to the Canadian Standards Association CSA-S37-01 Appendix M (CSA, 2001), the tower performance level and the seismicity of the site determine the appropriate level of detail of the seismic design check, as shown in Table 1. A simple classification is used to define the seismicity level, based on the peak horizontal ground acceleration, with the three categories of high (> 30 % g), moderate (15 – 30 % g) and low (< 15 % g). It is noted that in some cases the recommendations mention a static check for towers mounted on building rooftops, whereas no specific guidance is provided for such a static verification.

Structural dynamic principles dictate that the seismic sensitivity of a structure founded on ground is influenced by the coincidence between its dominant natural frequencies and the frequency content of the ground motion. Consequently, the first step to assess the tower sensitivity is to compare the range of natural frequencies of the tower and the frequency content of earthquakes. Historical earthquake records have typical dominant frequencies in the range of 0.1 – 10 Hz, with a concentration in the 0.3 – 3 Hz range for horizontal motion, while vertical motion involves frequencies higher than 3 Hz (Chopra, 2001). Vertical motion is not considered in this study, as the main point of interest lies in the investigation of building and tower horizontal response to earthquakes.

Table 1. Seismic Design Check Recommendations of CSA S37-01 (CSA, 2001)

<table>
<tr><th>Level of Seismicity</th><th>Life Safety</th><th>Interrupted Serviceability</th><th>Continuous Serviceability</th></tr>
<tr><td>Low Seismicity</td><td>• No seismic check necessary</td><td>• No seismic check necessary</td><td>• No seismic check necessary</td></tr>
<tr><td>Moderate Seismicity</td><td>• Static check for building-supported towers</td><td>• Static check for building-supported towers and irregular towers (geometry/mass)
• Dynamic check for masts of height 300 m and more</td><td>• Dynamic check for all tower types</td></tr>
<tr><td rowspan="2">High Seismicity</td><td>• Static check for all free standing towers of height 50 m and more and masts from 50 m to 150 m</td><td>• Static check for all free standing towers and masts up to 150 m</td><td rowspan="2">• Dynamic check for all tower types</td></tr>
<tr><td>• Dynamic check for masts of height 150m and more</td><td>• Dynamic check for masts of height 150m and more</td></tr>
</table>

Self-supporting lattice towers usually behave as geometrically linear structures and their dynamic response is simple to evaluate using modal superposition analysis. In Canada, short towers with a maximum height of about 50 m are typically structures of high fundamental frequencies and are not significantly affected by earthquakes. For towers in the 50 – 150 m height range, proportioning for bending and torsional rigidity (serviceability criteria) most often results in structures with natural frequencies lying in the sensitive range of 1-10 Hz. However, the effects of wind loads or combined wind and ice loads tend to be more critical than earthquake effects when strength is considered, partly because these climatic loads are amplified with load factors in the analysis, whereas the load factor applicable to the design earthquake is unity. These towers should therefore remain elastic during the design earthquake if interrupted or continuous serviceability is required.

The only simplified methods available to predict the seismic base shear demand of lattice towers built on rooftops can be found in building codes or related publications, in clauses devoted to non-structural building components. Numerical examples using provisions of the proposed 2005 National Building Code of Canada (NRC/IRC, 2005) are shown later for the cases modeled in this study.

Earthquake amplification factors for the base shear and the total vertical reaction of self-supporting telecommunication towers on firm ground were first suggested by Khedr and McClure (1999). Linear functions of tower mass, fundamental axial or lateral frequency and corresponding peak ground accelerations were established to estimate those values. However, it has been shown that base overturning moment is a better response indicator than base shear for these towers, and a simple predictor was suggested by McClure et al (2000). The goal of this study is to explore whether similar simplified indicators could be derived for towers mounted on rooftops.

MODELING CONSIDERATIONS

Tower models

For the simulations, two self-supporting three-legged steel lattice telecommunication towers – normally built on ground – are considered: Tower GSM30 is 30-m tall and Tower GSM40 is 40-m. A general view of their model geometry is shown in Figure 1. Both are modeled as linear elastic, three-dimensional frame-truss structures with frame elements for the main legs and truss elements for the diagonal and horizontal members. The supports are assumed pinned at the base. The mass of the main legs and their tributary members, which are bracing members spanning between the legs without any intermediate joint, are lumped at the corresponding leg joints. For truss members spanning between the legs with intermediate joints, the mass is summed up at the appropriate leg joints so that the member itself is assumed to be mass less. All secondary or redundant members, used for stability purposes, are removed from the stiffness model since they do not carry any load in a linear analysis. However, their mass is calculated and lumped at the corresponding leg joints. This mass lumping on the main members is done to avoid local spurious vibrations of the secondary members. It is noted that the mass of the bare towers is considered, i.e. without a provision for antennas, attachments and other secondary components. As a static load case, only the dead load is defined, and this load is combined with the earthquake load in the dynamic simulations.

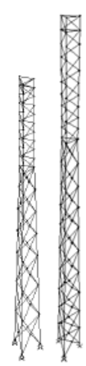

Figure 1. Geometry of Models of GSM30 (left) and GSM40 (right)

Frequency analysis of Tower GSM30

Figure 2 shows the lowest eight calculated natural frequencies and mode shapes of Tower GSM30. The fundamental sway mode is found at 2.69 Hz (repeated due to symmetry). The fundamental torsional mode (Mode 3) appears at a frequency of 9.16 Hz and is followed by the second sway modes in the X and Y directions (Modes 4 & 5) at 10.1 Hz. The second torsional mode is represented by Mode 6 at 18.3 Hz, which is outside the sensitive range. The number of mode shapes considered for modal superposition is chosen such as to include at least the lowest three sway modes of the tower, which spans up to 22 Hz in this example.

Frequency analysis of Tower GSM40

Although detailed results are not shown here, frequency analysis was also carried out on Tower GSM40. The fundamental sway mode was obtained at 2 Hz and consideration of the lowest three sway modes spans up to 15 Hz.

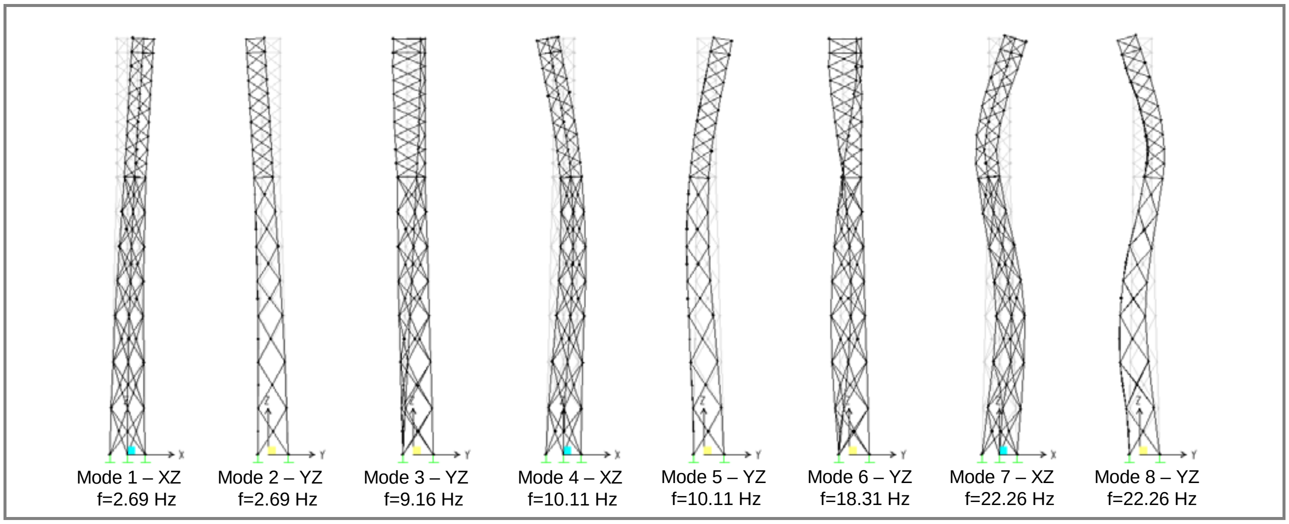

Figure 2. Mode Shapes and Natural Frequencies of GSM30

Building models

The first building to be investigated is Burnside Hall of McGill University in Montreal. This 14-story reinforced concrete building (Figure 3) is located at 805 Sherbrooke Street West and was built in 1970. The second building is a 27-story concrete frame building (Figure 4) located a few blocks from McGill Lower Campus at 2020 University Street. The construction of this building was completed in 1973.

Figure3: Burnside Hall

Figure 4: 2020 University

Burnside Hall model

Burnside Hall has a rectangular plan with lengths of 30.480 m (X-direction) and 33.528 m (Y-direction). Its total height is 51 m, with the first floor built at ground level. The lower levels, occupied by underground car garages, are assumed to be rigid. From the detailed structural drawings, effective storey mass and in-plane rotational inertia were evaluated for each floor (rigid diaphragm) and specified at the center of mass. A more precise mass distribution is used at Floor 13, a mechanical floor with several stiffness discontinuities and heavy equipment, and at roof level, to capture the eventual interaction with the tower mounted on the roof. Lateral and torsional stiffnesses of the columns, external structural walls and elevator shaft were obtained from the calculation of equivalent cross-sectional properties. The simplified model consists of six equivalent columns. A mesh of the model is shown in Figure 5. Thus, the building model has two translational, one vertical, and one in-plane torsional degrees of freedom on each floor. The masses represent the dead load, which is combined with the earthquake load in the dynamic analyses.

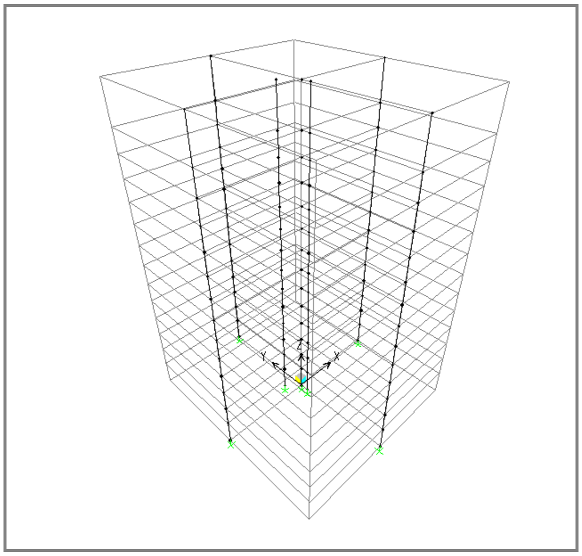

Figure 5. Burnside Hall Model

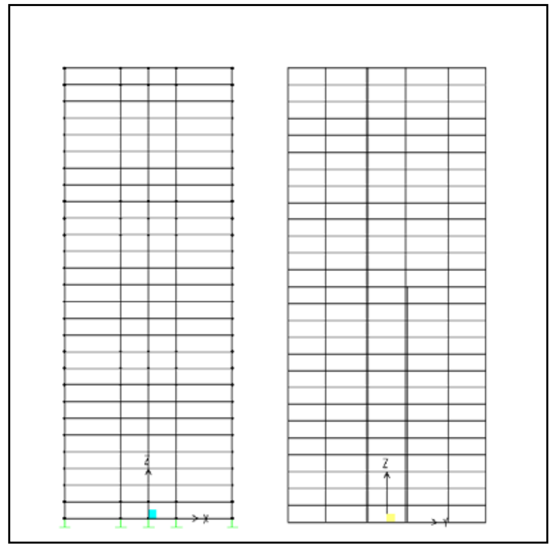

Figure 6. X-Z and Y-Z Projections of 2020 University Model

Mode shapes and natural frequencies of Burnside Hall

As for the towers, the mode shapes of Burnside Hall alone were calculated. The fundamental frequency is found at 2.21 Hz for the first sway mode in the Y direction, and at 2.30 Hz in the X direction, respectively (Modes 1 & 2). The first torsional mode appears at 3.74 Hz (Mode 3) and is followed by the second sway mode at 9.81 Hz in the Y direction and at 10.4 Hz in the X direction (Modes 4 & 5). The second torsional mode (Mode 6) follows at 16.5 Hz, and the third sway modes in the Y and the X directions are found at 20.9 Hz and 22.2 Hz, respectively (Modes 7 & 8). With these lowest eight modes, an effective participating mass ratio of 96 % is reached.

2020 University model

The 2020 University has a rectangular plan with dimensions of 36 m (X-direction) and 42 m (Y-direction) and a total height of 97.2 m. In general, the modeling assumptions for this building were similar to those of Burnside Hall. An important characteristic of the building is its asymmetry from the 15th floor up: two elevator shafts exist in the lower floors and only one in the upper levels (> 15th floor). As a result, the center of gravity changes at this intersection point and an eccentricity of the order of 0.33 m (with respect to the center of stiffness) is obtained for the mass in the upper floors. Except for the elevator shafts, the building has no shear walls. So in this case, all the columns are modeled as individual elements. A mesh of the model is shown in Figure 6.

Mode shapes and natural frequencies of 2020 University

After numerical trials of modal superposition analysis with different numbers of mode shapes, it was found that consideration of the 25 lowest modes was necessary, which spans the frequency range of 0.39-14 Hz. An effective participating mass ratio of 90% is obtained if at least 15 mode shapes are included, but Mode 15, at 7 Hz, still lies in the seismic range. The fundamental lateral frequency is found at 0.39 Hz for the sway in the X direction (Mode 1) and in the Y direction at 0.44 Hz (Mode 2). Mode 3 represents the first torsional mode with a frequency of 0.58 Hz. The second sway modes are determined for the Y direction at a frequency of 1.32 Hz (Mode 4) and for the X direction in combination with torsion at 1.71 Hz (Mode 6). The third sway in the Y direction is found for Mode 8 at a frequency of 2.81 Hz, and in the X direction – again combined with a torsional mode – for Mode 10 at 4.42 Hz.

In summary, the fundamental lateral frequencies of the tower models are 1.98 and 2.69 Hz, whereas the building models have 0.39 and 2.31 Hz. For towers mounted on rooftops, the buildings' frequencies will determine the frequency content of the excitation at the tower base. Therefore, both the buildings and the towers, considered separately as well as combined, are expected to be sensitive to earthquakes.

Combined tower-building models

The towers are assumed mounted at the position of the elevator shaft on the roof of the buildings. This position minimizes torsional effects on the building as a whole and on the top portion, in particular. As for the interface roof-tower in the model, a stiff, triangular plate is pinned to the three main tower legs, and the centroidal joint is merged into the core joint. The principal directions of the towers are coincident with the main axes (X-Y) of the buildings. Figure 7 shows the combined model for 2020 University and Tower GSM30, and Figure 8 shows the lowest frequencies and corresponding mode shapes of the combined model.

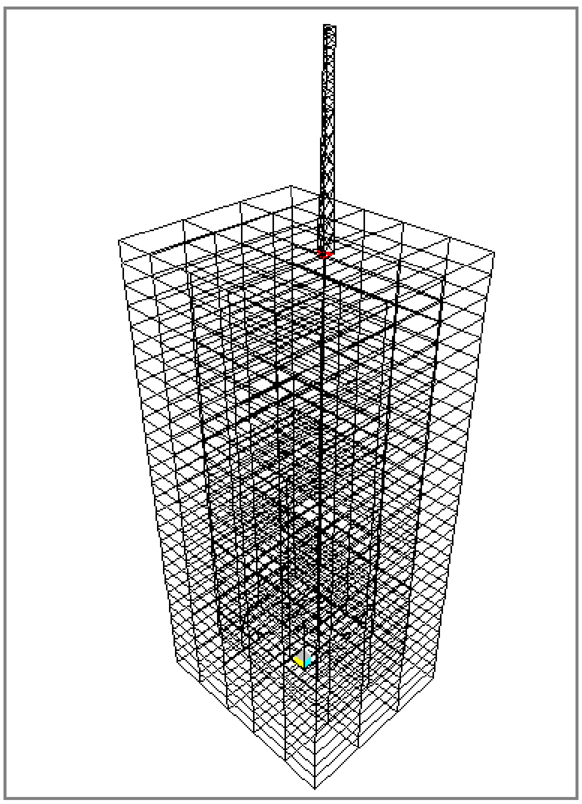

Figure 7. 2020 University with GSM30 Tower

For the four models studied, it was found that the presence of the tower has negligible influence (fourth significant figure) on the lowest three sway modes and frequencies of the building, contributing to a slight shift in the lower range due to the added mass at roof level. As expected, the influence of the building on the sway modes of the towers is very important, whereas torsional modes are not affected. Results are shown in Table 2 for the lowest three sway modes of the towers. The frequency shift in the lower range is even larger for 2020 University, which acts as a more flexible base (fundamental frequency of 0.39 Hz) than Burnside Hall (fundamental frequency of 2.2 Hz).

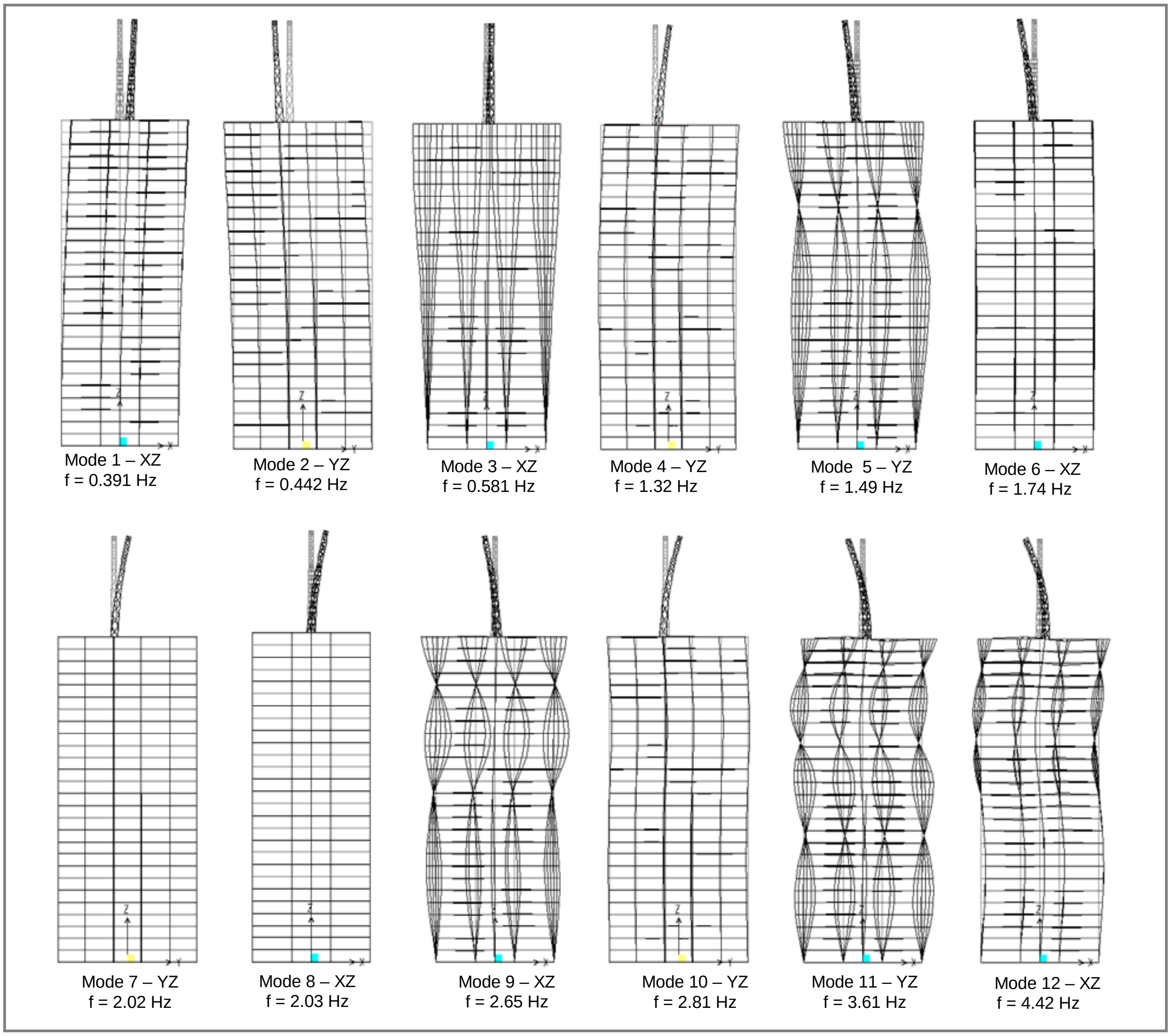

Figure 8. Lowest Natural Frequencies and Mode Shapes of 2020 University with GSM30

Table 2. Comparison of Natural Frequencies (Hz) of Telecommunication Towers

Modes	GSM30 on stiff ground	GSM30 & Burnside Hall	GSM30 & 2020 University	Modes	GSM40 on stiff ground	GSM40 & Burnside Hall	GSM40 & 2020 University
Sway 1	2.69	2.02	1.32	Sway 1	1.98	1.38	1.31
Torsion 1	9.16	9.16	9.16	Sway 2	7.03	5.67	2.65
Sway 2	10.1	8.5	2.81	Torsion 1	7.60	7.60	7.60
Torsion 2	18.3	18.3	18.3	Torsion 2	14.2	14.2	14.2
Sway 3	22.3	10.4	8.5	Sway 3	15.3	9.81	5.67

Input ground accelerations

The models of the two buildings alone and in combination with the two towers are subjected to a set of 45 historical earthquake records, which are fully described in Tso et al. (1992). This is the same set that was used by Khedr and McClure (1999) to derive simplified formulas of seismic amplification factors for lattice towers on stiff ground. The set contains both near-field and far-field records and the peak horizontal ground accelerations lie between 0.3 m/s^2 and 10.8 m/s^2. The records were further classified in three groups, in accordance with their maximum ratio of peak ground acceleration to peak ground velocity – a/v ratio – as follows: low with $a/v \leq 0.8$ g/(m/s), medium with 0.8 g/(m/s) $< a/v <$ 1.2 g/(m/s) and high with $a/v \geq 1.2$ g/(m/s). The ground acceleration is specified at the base of the building, with a magnitude of 100% of the record in each main axis of the building (X,Y) combined with 30% of the record in the perpendicular direction.

Seismic analysis

Truncated modal superposition with static correction and CQC modal combination was used in commercial software SAP2000 (1998) for all the numerical simulations. The number of mode shapes of the combined models to include in the simulations was selected so as to yield a participating mass ratio of at least 90%, and at least the lowest three sway modes of the modeled towers built on rooftops. For the Burnside Hall combined models, these criteria lead to 16 modes (up to 22 Hz and a participating mass ratio of 96%), and to 30 modes (up to 13.7 Hz and a participating mass ratio of 97%) for the 2020 University combined models. All materials are assumed linear elastic, and the motion is assumed in small kinematics. Modal viscous damping of 3% critical is used in each mode.

RESULTS

The results of the numerical simulations provide a data base to explore the correlation between various peak response indicators such as:

- rooftop acceleration vs. ground acceleration;
- tower top acceleration vs. rooftop acceleration;
- horizontal relative displacement between the tower top and base vs. rooftop acceleration;
- tower base shear vs. rooftop acceleration;
- tower base overturning moment vs. rooftop acceleration.

Some typical results are presented below. It should be mentioned that the abscissae in all graphs are horizontal accelerations corresponding to the main input direction, i.e. 100 % of the maximum ground acceleration of the earthquake records. It is understood that the calculated linear regression constants (m values) are problem specific, and the objective here is to identify whether simple linear predictors are good candidates to provide reliable results.

Rooftop and tower accelerations

Figure 9 shows the relation between rooftop acceleration and ground input acceleration for the three Burnside Hall models: the building alone and the building combined with GSM30 and GSM40. If the two results with high acceleration (above 10 m/s^2) are omitted, the linear trend shows a coefficient of correlation, R^2, of 0.76 for loading in the main X direction, and 0.81 for the main Y direction (not shown). The coefficient for 2020 University is 0.72 in both cases.

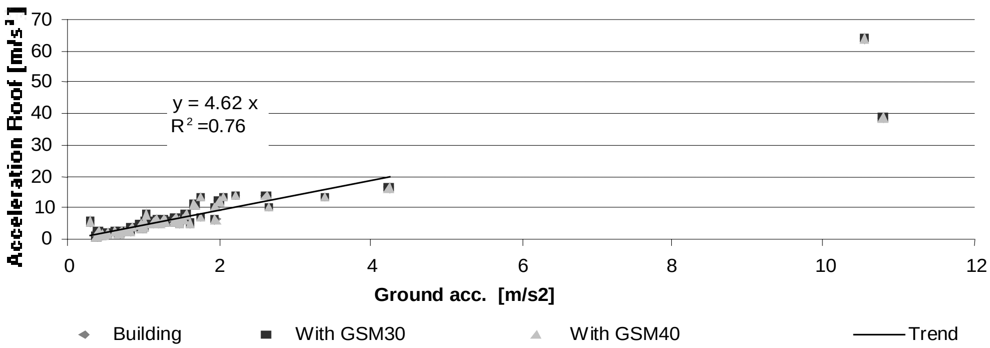

Figure 9. X-Acceleration Rooftop - Burnside Hall – Ground (main input X)

Considering that variations are very small between the results of the three models, one can continue the comparisons with the rooftop acceleration for the building alone. The coincidence of the results for the different models is explained by the relatively small mass of the towers compared to the building. Table 3 shows the ratios of the masses of GSM30, GSM40, and the top level of each building to the total building mass.

Table 3. Tower/Building Mass Ratios

	Burnside Hall			2020 University		
	Mass [tons]	Ratio to Building [%]	Ratio to Roof [%]	Mass [tons]	Ratio to Building [%]	Ratio to Roof [%]
Building	12 590	100	-	30 795	100	-
Roof Level	275	2.2	100	1 050	3.4	100
GSM30	2.25	0.018	0.82	2.25	0.007	0.21
GSM40	3.59	0.029	1.31	3.59	0.012	0.34

Figure 10 shows the relation between the acceleration at the top of the tower and the acceleration at roof level in the X direction due to the main input ground motion also in the X direction, for the Burnside Hall & GSM30 pair. The linear trend is clearly confirmed, with more dispersion on the large intensity accelerations. A summary of the results is presented in Table 4.

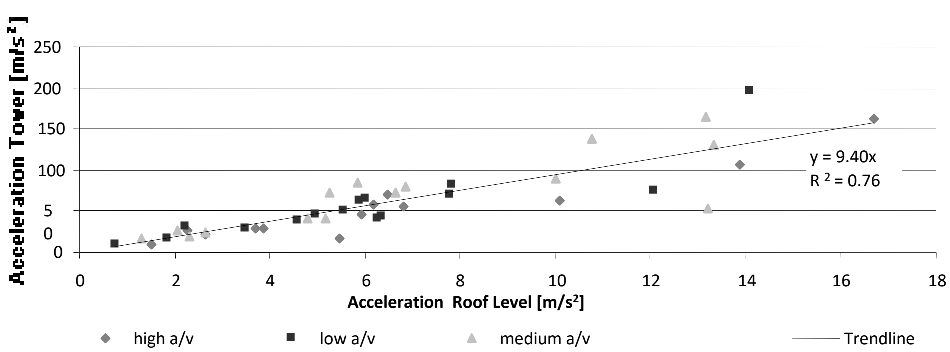

Figure 10. X-Acceleration GSM30 – Burnside Hall (main input X)

Theses results indicate that it is possible to get a good linear predictor of the tower accelerations based on rooftop values, considering the coefficient of correlation R^2 is 0.71 and above. They also show the large amplification of the acceleration due to dynamic response of the towers: rooftop towers cannot simply be considered as rigid bodies undergoing the same acceleration as the roof. This will be further evidenced by the results obtained for the tower base shear.

Table 4. Regression Parameters: Acceleration at Tower Top vs. Rooftop Acceleration ($A_{tower} = m A_{roof}$)

Model	Parameter	A_x, X[(i)]	A_x, Y	A_y, X	A_y, Y
Burnside Hall & GSM30	m	9.40	9.37	11.6	11.6
	R^2	0.76	0.76	0.81	0.81
Burnside Hall & GSM40	m	3.47	3.48	3.63	3.67
	R^2	0.73	0.73	0.85	0.84
2020 University & GSM30	m	10	10	7.4	7.4
	R^2	0.71	0.71	0.76	0.76
2020 University & GSM40	m	8.3	8.3	11	11
	R^2	0.75	0.75	0.52	0.51

[(i)] Notation: Tower acceleration along X due to main input along X

Values of parameter "m" need to be linked to the dominant frequencies of the roof acceleration (related to ground input and building response) and the lowest frequencies of the towers. For example, for Burnside Hall, the accelerations of GSM30 are more amplified than those of GSM40. This is explained by the fact that the dominant frequencies of GSM30 mounted on the roof (2.02 Hz for Modes 1 & 2) are closer to the fundamental sway mode of the building (2.2 Hz) than those of GSM40 (1.38 Hz for Modes 1 & 2). Assuming all the earthquake records used in the study excite the building's fundamental sway mode, the acceleration at the roof level is dominated by the fundamental sway mode of the building. As a result, a tower with frequencies close to that fundamental sway mode will see its response much amplified. In a situation where the building's fundamental sway mode is not excited by the ground motion, the amplification of the motion at the rooftop is not as important unless there is coincidence with higher sway modes. It is also observed that the trend is similar for the two main input directions of the ground motion, indicating that there is not much coupling between the two main directions in the response of the roof at the tower base.

Tower base shear

Figure 11 shows a typical relation between the peak value of the tower base shear and the peak rooftop acceleration in the Y direction due to an input ground motion with main component in Y, for the 2020 University & GSM30 pair. The linear trend is not as strong as in the case of peak acceleration at the tower top, and the dispersion is no more confined to the range of large intensity accelerations. A summary of the regression parameters obtained is presented in Table 5.

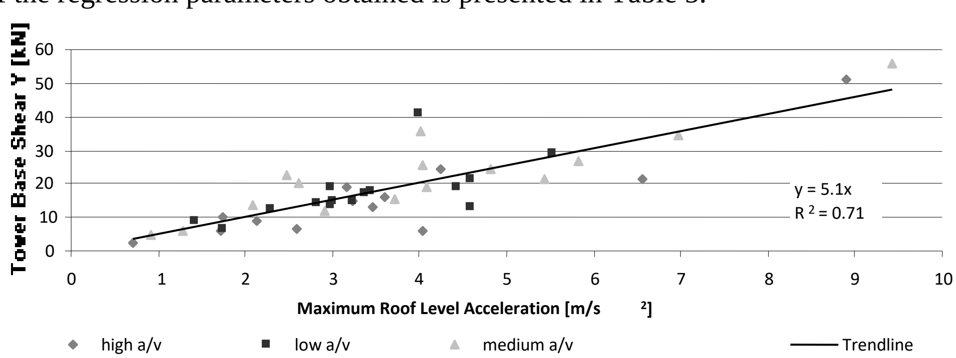

Figure 11. Base Y-Shear GSM30 – 2020 University (main input Y)

The peak response in base shear is consistent with the peak accelerations in Table 4.

Table 5. Regression Parameters: Tower Base Shear vs. Rooftop Acceleration ($V_{base} = m A_{roof}$)

Model	Parameter	V_x, X[(i)]	V_x, Y	V_y, X	V_y, Y
Burnside Hall & GSM30	m	6.5	6.5	8.4	8.4
	R^2	0.72	0.74	0.77	0.76
Burnside Hall & GSM40	m	1.8	1.8	2.1	2.1
	R^2	0.26	0.26	0.51	0.48
2020 University & GSM30	m	8.6	8.7	5.1	5.1
	R^2	0.67	0.66	0.71	0.71
2020 University & GSM40	m	7.6	7.7	12	12
	R^2	0.72	0.69	0.49	0.46

(i) Notation: Tower base shear along X due to main input along X

Considering that the mass ratio between GSM30 and GSM40 is 0.63, results show that there is significant dynamic interaction between the towers and the buildings. For a better illustration of this effect, Table 6 presents the base shear values normalized with respect to the weight of the towers.

Table 6. Regression Parameter m: Normalized Base Shear vs. Rooftop Acceleration ($V_{base}/W = m A_{roof}$)

Model	V_x, X[(i)]	V_x, Y	V_y, X	V_y, Y
Burnside Hall & GSM30	0.29 (0.65) [(ii)]	0.29	0.38	0.38 (2.2)
Burnside Hall & GSM40	0.05 (0.65)	0.05	0.06	0.06 (2.2)
2020 University & GSM30	0.40 (0.52)	0.39	0.23	0.23 (0.48)
2020 University & GSM40	0.22 (0.52)	0.22	0.34	0.34 (0.48)

[(i)]Notation: Tower base shear along X due to main input along X
[(ii)] Values in parentheses are predictions using NBCC 2005

Comparison with predictions of NBCC 2005

The values predicted by NBCC 2005 are obtained with the following equation (from section 4.1.8.17):

$$V_p = 0.3 F_a S_a(0.2) I_E \frac{C_p A_r A_x}{R_p} W_p$$

The expression 0.3 F_a S_a (0.2) I_E represents the peak acceleration at the base of the building, $C_pA_rA_x/R_p$ is the amplification factor of the base acceleration, and W_p is the weight of the component.

C_p is a component factor, taken as 1.0 for towers, which accounts for the risk to life safety associated with failure of the component. A_r is the component force amplification factor: it is function of the ratio of the fundamental period of the component and the fundamental period of the building, and varies from 1.0 to 2.5. If the period of the component is not known it is assumed equal to 2.5. In the examples studied, the value of A_r is 2.5 for Burnside Hall and 1.0 for 2020 University, for either tower. A_x is the height factor (taken as $1+2h_x/h_n$), which assumes a linear amplification of acceleration through the height of the building. For the rooftop ($h_x = h_n$), $A_x = 3$. R_p is the component response modification factor and accounts for the energy-absorption capacity of the component and its connection to the building framework. It varies from 1.25 to 5 and for lattice towers in the post-elastic regime, $R_p = 2.5$.

Finally, the coefficient $C_pA_rA_x/R_p$ is 3.0 for Burnside Hall and 1.2 for 2020 University. It is important to note that the value of the slope parameter, m, in Table 6, refers to the peak roof acceleration, so it is necessary to adjust the calculation of the value predicted by the code by the ratio of roof-to-ground peak accelerations. It is seen that the code predictions are systematically conservative, especially in the case of Burnside Hall, but inconsistent with the simulation results.

Tower overturning moment

Figure 12 shows the relation between the peak tower base overturning moment and the peak rooftop acceleration in the X direction due to an input ground motion with main component in the Y direction, for the 2020 University & GSM30 pair. The trend is similar to that obtained for the base shears with dispersion in the whole range of roof accelerations considered. A summary of the regression parameters obtained is presented in Table 7.

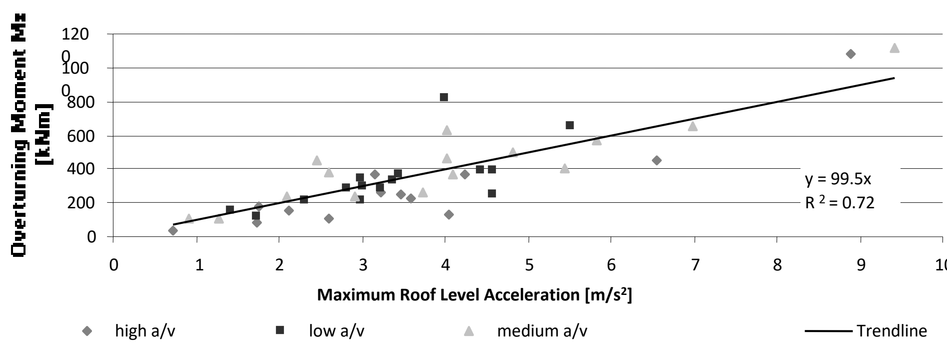

Figure 12. M_x at GSM30 Base – 2020 University (main input Y)

Table 7. Regression Parameters: Tower Base Moment vs. Rooftop Acceleration ($M_{base} = m\ A_{roof}$)

Model	Parameter	M_x, X(i)	M_x, Y	M_y, X	M_y, Y
Burnside Hall & GSM30	m	171	170	136	137
	R^2	0.76	0.75	0.73	0.76
Burnside Hall & GSM40	m	55	56	49	49
	R^2	0.61	0.59	0.49	0.49
2020 University & GSM30	m	100	99.5	161	163
	R^2	0.73	0.72	0.64	0.64
2020 University & GSM40	m	259	258	165	167
	R^2	0.49	0.46	0.65	0.62

(i) Notation: Tower base moment about X (caused by shear in Y) due to main input in X

The results obtained for the overturning moments do not show strong linear correlation with the rooftop peak accelerations, especially for the GSM40 tower. This indicates that a simple linear predictor would not be satisfactory for taller, more flexible towers on rooftops.

CONCLUSION

The objective of this work was to study the seismic response of two different lattice towers mounted on the rooftop of two medium-rise buildings (Burnside Hall and 2020 University, located in Montreal, Canada). The aim was to find whether simple linear relations could represent the variation of the tower response as a function of the peak roof acceleration.

Since the presence of a normal size tower on the buildings does not have a significant influence on the building frequencies and mode shapes, it is desirable to use a prediction of the roof acceleration based on the natural frequencies of the building alone. However, the natural frequencies of the towers are much affected by the flexible base provided by the building and it is necessary to find a simple rational way to predict this frequency shift.

Moreover, a linear correlation between the rooftop acceleration and the tower base reaction – shear force and overturning moment – was obtained with reasonable accuracy for a design check of the base connection, for the models in combination with tower GSM30. Also with the model of 2020 University in combination with GSM40, representative equations could be established. Yet, the model combining Burnside Hall and tower GSM40 represented inconsistencies for these functions. It should be mentioned that GSM40 is very flexible and tall compared to the buildings and would not represent a realistic example of most towers on rooftops.

Nonetheless, a series of simple linear predictors seem promising. Future work will attempt to quantify such predictors, based on: peak roof acceleration, tower-to-building frequency ratio, tower weight and height (for the overturning moment predictor). These predictors will need validation with more building and tower models covering different heights, and stiffness and mass properties.

ACKNOWLEDGMENTS

Financial support from the Natural Sciences and Engineering Research Council of Canada and the Lebanese Research Council is gratefully acknowledged. Burnside Hall data was provided by McGill University. Tower data was provided by the International Association For Shells and Spatial Structures (IASS) Working Group No. 4 on Masts and Towers.

REFERENCES

1. Chopra AK (2001). *Dynamics of Structures – Theory and Application to Earthquake Engineering*. Prentice Hall.
2. CSA S37-01 (2001). Antennas, Towers, and Antenna-Supporting Structures. Appendix M "Earthquake- Resistant Design of Towers", Canadian Standards Association, Rexdale, Ontario, Canada.
3. Khedr MA and McClure G (1999). Earthquake Amplification Factors for Self-supporting Telecommunication Towers, *Canadian Journal of Civil Engineering*. Vol. 26 No. 2, 208-215.
4. McClure G, Lapointe M and Khedr MA (2000). Seismic Behaviour of Steel Lattice Telecommunication Towers. *STESSA 2000 Behaviour of Steel Structures in Seismic Areas*. Third International Conference, Montreal, Canada, 21-24 August, 335-337.
5. NRC/IRC (proposed 2005). *National Building Code of Canada, 2005*, 12th Edition; National Research Council of Canada, Institute for Research in Construction, Ottawa, Canada.
6. SAP2000 (1998): *Basic Analysis Reference*. Integrated Finite Element Analysis and Design of Structures. Computers and Structures, Inc. Berkeley, California, USA Version 7.0.
7. Tso WK, Zhu TJ and Heidebrecht, AC (1992). Engineering implication of ground motion A/V ratio. *Soil Dynamics and Earthquake Engineering*, 11(3), 133-144.