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# SEISMIC FORCE AMPLIFICATION FACTORS FOR TELECOMMUNICATION TOWERS MOUNTED ON BUILDING ROOFTOPS

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## ABSTRACT

Telecommunication towers mounted on building rooftops are considered as flexible non-structural components from a building code perspective. In Canada, these are known as Operational and Functional Components (OFCs). Due to limited research on the seismic performance of OFCs, the current building code guidelines for seismic design of rooftop telecommunication towers are usually adequate only for very stiff towers. However, the behavior of flexible towers is different and this paper is an attempt to provide guidance on the prediction of the amplification of acceleration along the towers. Numerical simulations for four telecommunications towers of different heights (10-30 m) and geometric properties will be presented. These towers are assumed mounted on four buildings having different heights and lateral load resisting systems. The detailed finite element model of each building-tower combination was subjected to two large sets of earthquakes: one set of records is compatible with the target uniform hazard spectra of the 2005 National Building Code of Canada (NBCC) for Montreal, and the other set comprises 44 records classified according to their peak ground acceleration-to-velocity (a/v) ratio.

## Introduction

A building is made up of various components that can be divided into two groups: structural components and operational and functional components (OFCs). According to CSA S832-01 (CSA 2001), operational and functional components are those systems and elements housed in or attached to floors, roofs, and walls of a building or industrial facility, and that are not part of the main or intended load bearing structural system. However, these components may contribute to the structural integrity of a building, depending on their location, type of construction, and method of fastening. Operational and functional components can generally be divided into three sub-components according to CSA S832-01 (CSA 2001) and Villaverde

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(1997): Architectural, internal and external, like cladding, interior partition walls, ceilings and lights; building services including mechanical and electrical systems, such as electrical power distribution systems, heating, ventilation, cooling systems, telecommunications; and building contents, such as computer systems.

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; Kao et al. 1999; Naeim 1999, 2000; Filiatrault et al. 2001) highlights the fact that the poor performance of OFCs is the greatest contributor to damage, losses, and business interruption in most essential and critical facilities.

In this article, we are mainly concerned about the seismic analysis of telecommunication towers that are required to remain functional during and after the earthquake and that are mounted on building rooftops

**Seismic provisions for telecommunication towers**

Telecommunication towers mounted on building rooftops are considered, from a building code perspective, as acceleration-sensitive flexible non-structural components. A flexible component can be identified as having a fundamental period of vibration greater than 0.06s. Current seismic provisions in codes and standards for telecommunication towers relate to structures on ground and are not applicable to towers on rooftops. The design of these towers is typically controlled by extreme wind, ice and wind combinations, restrictive deflection and rotation limits (TIA/EIA 222-G 2002, CSA-S37 2001); therefore, codes and standards are concerned with wind and ice loads in cold regions. However, when the tower supports heavy attachments in the upper portion 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 in active seismic zones.

The 2005 edition of the National Building Code of Canada (NRC/IRC 2005) proposes an empirical formula for the determination of seismic design base shear forces for telecommunication towers considered as OFCs attached to a building structure. However, guidelines to compute the distribution of lateral forces along the tower height and overturning moments at the tower base do not exist. The base shear formula as given in equation 1 is quite empirical and needs revision.

$$V_p = 0.3 F_a S_a(0.2) I_E S_p W_p \tag{1}$$

Where:

$0.3F_aS_a(0.2)I_E$ represents the input ground acceleration to the building with:

$F_a$ : acceleration-based site coefficient.

$S_a(0.2)$ : spectral response acceleration value at a period of 0.2s.

$I_E$ : importance factor for the building, varying between 1.0 for usual use and occupancy and 1.5 for post-critical facilities.

$S_p$ : $= C_pA_rA_x/R_p$ represents the component response amplification, varying between 0.7 and 4.0, with:

$C_p$ : component factor. It considers the risk to life safety associated with failure of the component and/or release of contents. It may vary from 0.7 to 1.5.

$A_r$ : component force amplification factor. It represents the dynamic amplification of the component relative to the position of its attachment to the building structure. It is function of the ratio of the fundamental period of the component (Tp) and the fundamental period of the structure (T) as shown in Figure 1. In case the ratio of the periods is not known, values are suggested for various component types; a value of 2.5 is given for towers.

$A_x$ : height factor. It considers the linear amplification of acceleration along the height of the building and is equal to (1+ 2hx/hn), in which $h_n$ is the total height of the building and $h_x$ is the floor elevation where the component is located.

$R_p$ : component response modification factor. It represents the energy-absorption capacity of the element and its attachment. It may vary from 1.25 to 5, and a value of 2.5 is suggested for towers.

Finally, $W_p$ is the weight of the component.

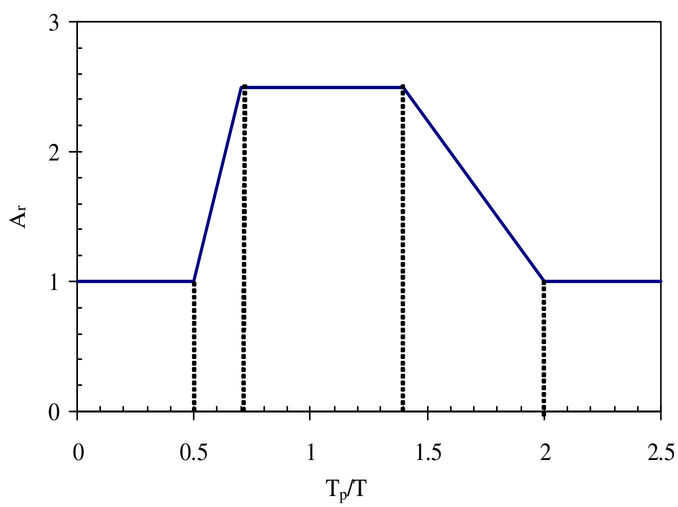



Figure1. Component force amplification factor according to NBCC 2005

The component force amplification $A_r$ will be discussed based on the results of detailed numerical simulations for building-tower models.

## Models used in this study

In this research, detailed numerical simulations were carried out for four buildings and four towers. Some geometric details of the buildings and towers as well as a brief overview of the main finite element modeling assumptions are presented next.

### Buildings

Some general properties of the four buildings studied are given in Table 1. The isometric and elevation views of the buildings are shown in Figures 2 to 5. The buildings range from low-rise to high-rise and from old to relatively recent constructions. LLRS refers to the lateral load resisting system. Frame refers to reinforced concrete frame system and dual refers to moment-wall system.

Table 1. Properties of the buildings studied

| Location | ID | Year of construction | Use | LLRS | No. of stories above ground | Height (m) |
|---|---|---|---|---|---|---|
| Tainan | CHYBA9 | 1980 | Telecom | Dual | 4 | 20 |
| Jia-Yi | CHYBA4 | 1983 | Hospital | Frame | 6 | 24.2 |
| Hsinchu | TCUBAA | 1996 | Library | Frame | 8 | 30.4 |
| Montreal | 2020 Univ. | 1973 | Office | Frame | 27 | 115.2 |

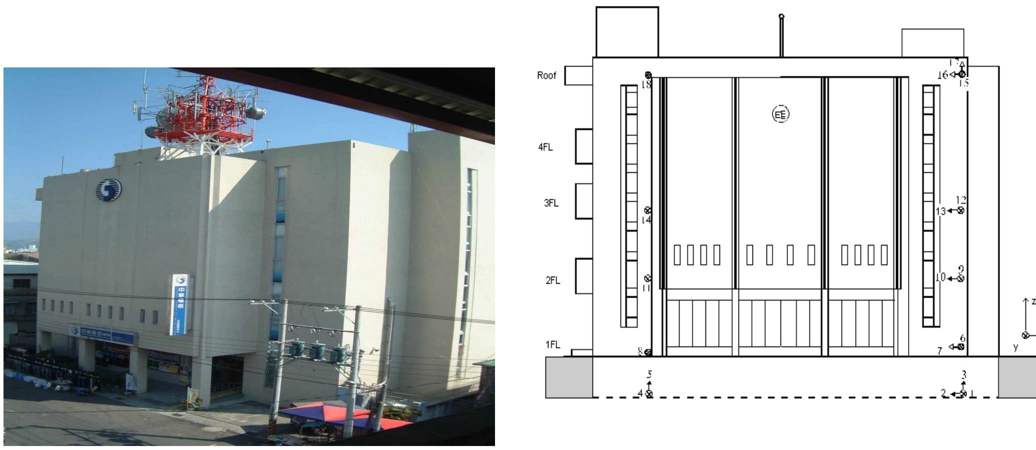



Figure 2. Isometric and facade elevation views of the CHYBA9 building

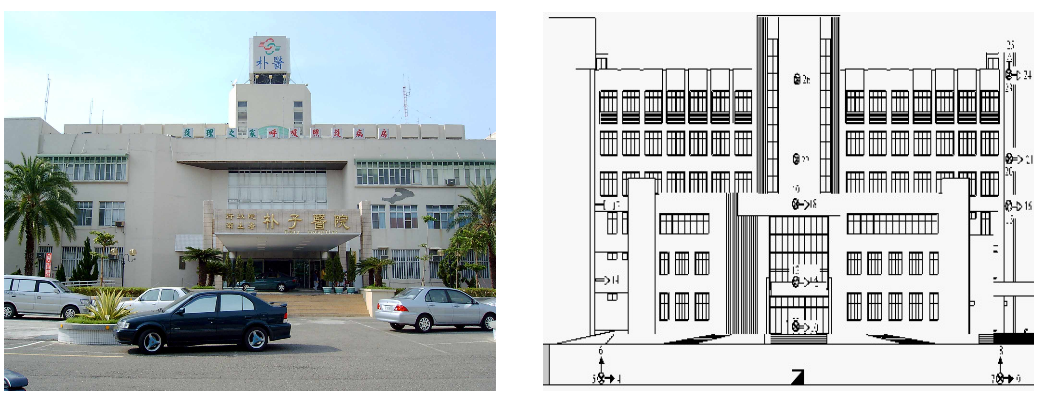


Figure 3. Isometric and facade elevation views of the CHYBA4 building

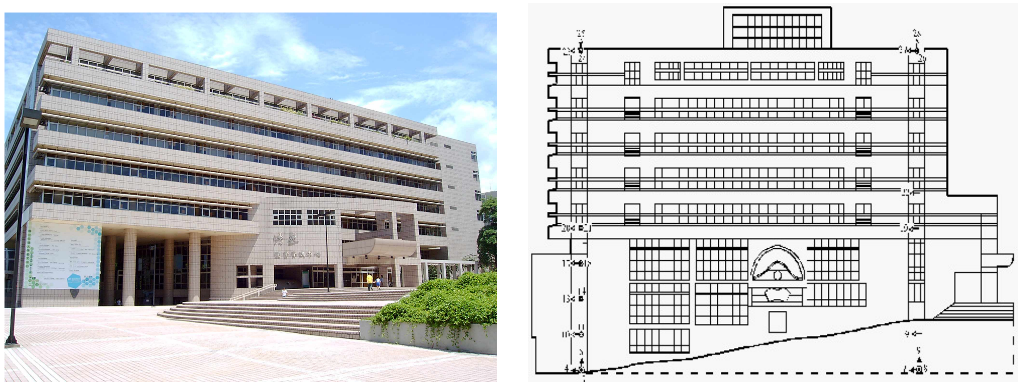


Figure 4. Isometric and elevation views of the TCUBAA building

Figure 5. Isometric view of the 2020 University building

Detailed three-dimensional elastic models of the four buildings were generated in SAP 2000. Rigid floor slabs were assumed, while the columns, beams, and walls were modeled in detail. The mass of non-structural components and finishing was distributed to columns and walls according to their tributary area. Table 2 summarizes the lowest periods of vibration obtained for the building models, corresponding to the fundamental sway modes and the torsional mode.

Table 2. Natural periods of the building models

| **ID** | **$T_1$ (s)** | **$T_2$ (s)** | **$T_3$ (s)** |
|---|---|---|---|
| CHYBA9 | 0.30 | 0.26 | 0.17 |
| CHYBA4 | 0.41 | 0.31 | 0.23 |
| TCUBAA | 0.75 | 0.69 | 0.62 |
| 2020 Univ. | 2.0 | 1.9 | 1.36 |

**Towers**

Three typical medium-height towers and one low-height rigid tower were studied. Table 3 summarizes the geometric properties of the towers, including their height, base width, top width, and mass. The mass of the towers doesn't include the mass of attachments and ladders except for the TC1 tower. The tower labeled TC1 is a 4-legged steel lattice tower having a square base, while the towers labeled TC2, TC3 and TC4 are 3-legged steel lattice towers with equilateral triangular base. The towers were modeled in Sap 2000 as three-dimensional frame-truss linear elastic structures. Frame elements were used for the main legs and truss elements for diagonal and horizontal members. The tower models were assumed rigidly connected to the roof of the building models. An example of such a connection is illustrated in Figure 6 for the telecom building in Tainan City. Figure 7 shows the finite element meshes of the tower models and Table 4 gives the four largest natural periods calculated.

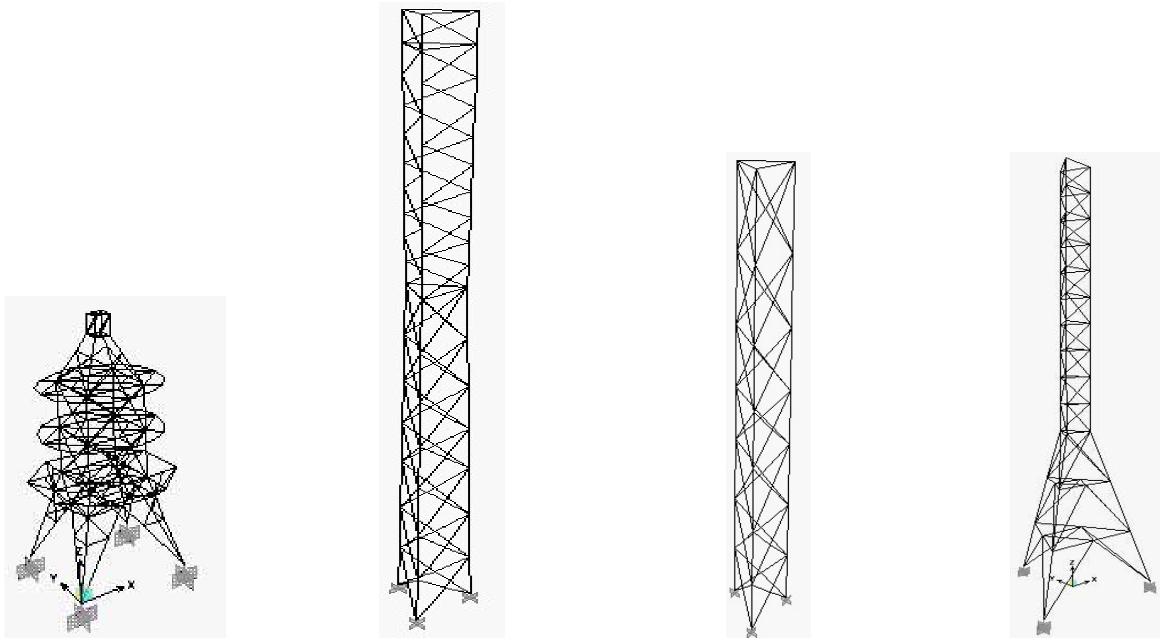
Table 3. Geometric properties of the telecommunication towers

| **Tower ID** | **Type** | **Height (m)** | **Base Width (m)** | **Top Width (m)** | **Mass (kg)** |
|---|---|---|---|---|---|
| TC1 | 4-Legged | 10.73 | 4.70 | 0.70 | 9566 |
| TC2 | 3-Legged | 30 | 2.50 | 1.50 | 2245 |
| TC3 | 3-Legged | 20 | 2.50 | 1.50 | 1735 |
| TC4 | 3-Legged | 20 | 5.50 | 1.30 | 2920 |

Figure 6. Tower base support for the TC1 tower

Table 4. Natural periods of the tower models

| **ID** | **$T_1$ (s)** | **$T_2$ (s)** | **$T_3$ (s)** | **$T_4$ (s)** |
|---|---|---|---|---|
| TC1 | 0.138 | 0.135 | 0.095 | 0.095 |
| TC2 | 0.372 | 0.372 | 0.109 | 0.099 |
| TC3 | 0.186 | 0.186 | 0.081 | 0.049 |
| TC4 | 0.254 | 0.254 | 0.084 | 0.048 |



a) TC1 b) TC2 c) TC3 d) TC4

Figure 7. Finite element meshes of the tower models

## Modal analysis

Using the 3-D finite element models generated in Sap 2000, elastic time-history modal superposition analysis was performed for each building-tower combination. 20 modes were considered and a uniform damping ratio of 3% was used for all modes. The models of the Chinese buildings were calibrated using the recorded floor accelerations from the 1999 Chi Chi earthquake and the fundamental periods extracted by system identification techniques using the same records. The combined building-tower models were subjected to two sets of earthquake records applied in both principal horizontal directions U1 and U2.

### Earthquake records

The first set of records comprises 44 historical records resulting from 23 events, classified into three categories according to the ratio of the peak ground horizontal acceleration to the peak ground horizontal velocity (high, medium and low a/v). More details about these earthquake records can be found in Tso et al. (1992). The second set comprises three series of ten generated time histories compatible with the target uniform hazard spectra (UHS) of Montreal city, Canada, corresponding to probabilities of exceedance of 2%, 10% and 50% in 50 years, respectively. These time histories were generated based on a stochastic approach presented in Atkinson et al. (1998). A total of 15 magnitude-distance (M-R) scenarios were applied to cover the entire frequency range of interest. Due to the randomness of the generated records, two acceleration time histories were used for each M-R scenario.

**Component force amplification factor**

In the absence of knowledge on the dynamic properties of the building and/or tower, the NBCC proposes a component force amplification factor, $A_r$, equal to 2.5 for all towers (NBCC 2005 Table 4.1.8.17). In case the ratio $T_{tower}/T_{builing}$ is known, the factor $A_r$ can be calculated from the graph shown in Figure 1. In order to investigate the adequacy of the $A_r$ factor proposed in the NBCC, the absolute seismic response accelerations along the tower height were estimated for each building-tower combination, in both orthogonal directions U1 and U2. The average value of acceleration amplification at different levels, from the tower-building interface (roof level) to the top of each tower, was computed. This average acceleration amplification is equivalent to the component force amplification factor $A_r$ suggested in the NBCC 2005. The graphs showing the average and standard deviation of amplification factors resulting from the numerical simulations for each building-tower combination are presented in Figures 8 to 11. The amplification profile as suggested in the NBCC 2005 (Fig. 1) is also shown on the graphs for comparison.

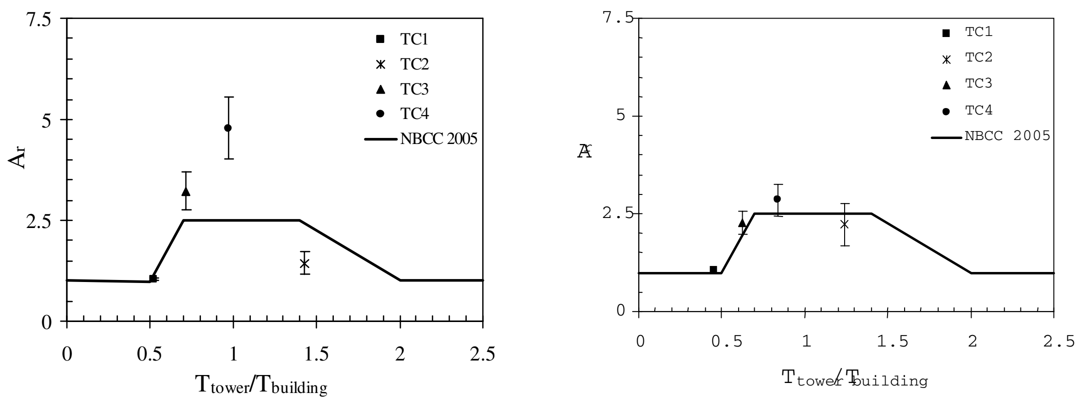



Figure 8. $A_r$ for CHYBA9 in the U1 and U2 directions

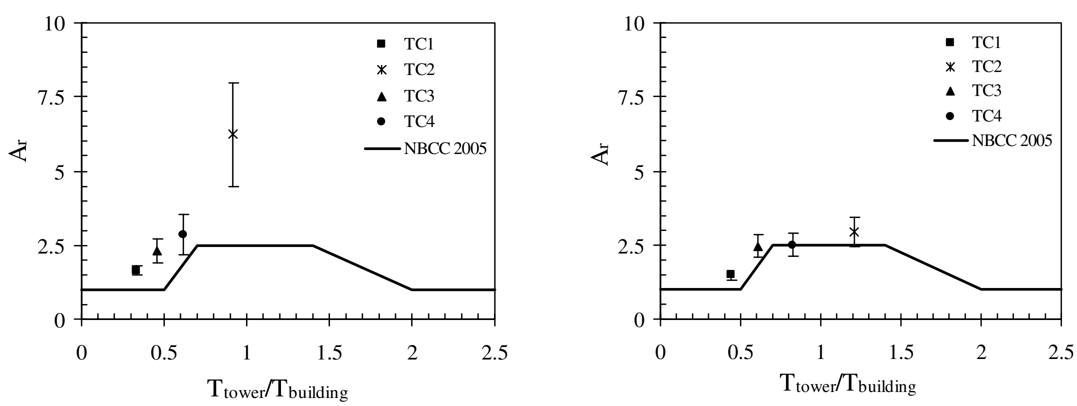



Figure 9. $A_r$ for CHYBA4 in the U1 and U2 directions

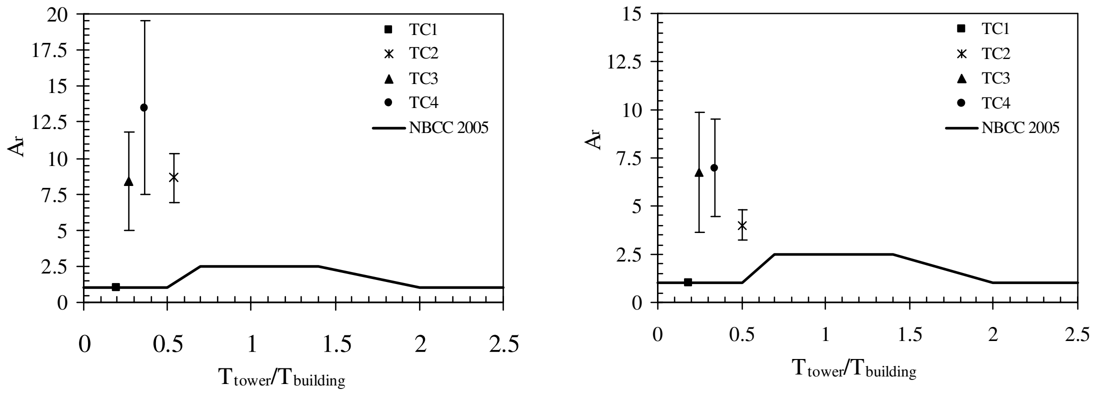



Figure 10. $A_r$ for TCUBAA in the U1 and U2 directions

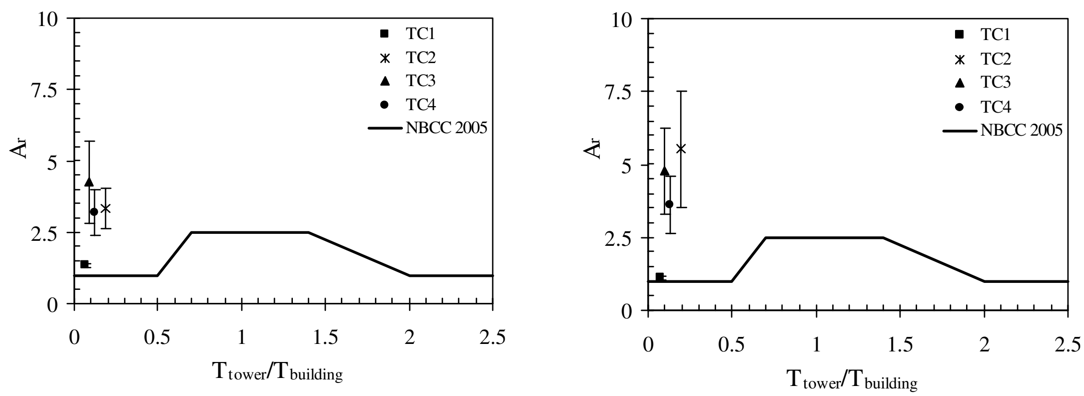



Figure 11. $A_r$ for 2020 University in the U1 and U2 directions

## Conclusions

For low-rise buildings, such CHYBA9 (Fig. 8) and CHYBA4 (Fig. 9), the component force amplification factor proposed in the NBCC 2005 gives reasonable results, especially for relatively rigid towers such as TC1. However, when the tower period approaches the building period, a factor of 6 seems more adequate. In case the period of the tower exceeds the period of the building such as the TC2 tower mounted on the CHYBA9 building, the factor of 2.5 proposed by NBCC seems reasonable. In case of medium and high-rise buildings, such as TCUBAA (Fig. 10) and 2020 University (Fig. 11), the amplification factor equal to 1 seems reasonable for the very rigid towers, such as the TC1 tower; however, in case of more flexible towers, it is suggested to increase the value of $A_r$ from 2.5, as proposed in NBCC 2005, to 8. Also, it is noted that for the TC1 tower, the amplification of acceleration is negligible for all buildings models, except for the CHYBA4 building, indicating that the NBCC limit of 0.06 s set for the fundamental period of rigid components is conservative. Also, we can notice that the standard deviation in the component force modification factor due to earthquake records increases as the building becomes more flexible.

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