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# Seismic demand and experimental evaluation of the nonstructural building curtain wall: A review

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ABSTRACT

This paper reviews the existing studies related to seismic demand and experimental investigation of the nonstructural building curtain wall (CW). Being treated as a nonstructural component, seismic performance of the building CW relates to its seismic demand parameters, i.e., acceleration and drift demands. In current code provisions, the acceleration demand consists of the floor acceleration amplification factor, component acceleration amplification factor, component importance factor, and component response modification factor, which are all based on or induced by the floor response and dynamic response of the CW itself. For the CW which is attached to the main structure, drift demand is an indication of the interstory drift ratio. The in-plane seismic drift mechanism of the framed glass CW was fully developed, and the corresponding static testing protocols were implemented in codes based on several past experimental studies. Shaking table testing of the CW was conducted as well, where the input motions need specific floor response analysis of the main structure. The relevant damage state definition and fragility curve development are important to represent the performance and damage level of the CW system. The philosophy of the performance-based earthquake engineering (PBEE) and its application to CW are elaborated, and possible challenges related to the seismic demand, experimental studies, and PBEE of the CW are addressed as well.

## 1. Introduction

Curtain wall (CW) indicates any building wall, of any material (e.g. stone, reinforced concrete, glass, metal, etc.), which carries no superimposed vertical loads, i.e., any 'nonbearing' wall. Building CW or façade provides the aesthetic, environmental, and structural functions to achieve the enclosure required for the safety, comfort and functionality of building occupants and contents. Precast reinforced concrete (RC), stone, and masonry are materials typically used in building facade. Due to the rapid development of manufacture and construction technologies, glass, steel, or natural stone panels are easily produced with lower cost than before, thus light and aesthetic façades are very popular in modern, especially tall and landmark, buildings. There are several terminologies used for the building façade, such as cladding, building envelope, and curtain wall. In this paper, however, curtain wall is used. Nowadays, compared to the traditional CW (e.g., masonry or stone CW), glass CW is light in mass, transparent, and relatively energy efficient. Therefore, glass CW became the most popular façade type.

In some countries (e.g., China), CW was architecturally conceived by the architect who is usually not involved in the structural design. Therefore, the load bearing or transfer profile of these CWs are not typically considered during the structural design. In practice, the construction industry usually conducted design, fabrication and installation of the CW in the building structure, with little or no consideration of structural analysis. Therefore, the structural performance of the CW would not be ensured. Nowadays, the CW design codes have been enforced where the CWs are designed by the CW designer company, the architect, or the structural engineer. The glass thickness, material, configuration, framing, and connections have to be designed for gravity and lateral wind/seismic loads. Furthermore, compared to the traditional structural elements, e.g., columns, slabs and girders, CW is relatively a novel subsystem in the modern building structural system, and the real earthquake damage observations are still rare. Although the seismic performance of CW system has become a greater concern to engineers due to the numerous CW failure cases in the past decades, seismic safety of the substantial amount of CWs cannot be verified by real earthquakes, even if they are carefully designed by qualified structural engineers. Study on the seismic performance of the CW system is scarcely reported in the literature. However, although CW is

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treated as a nonstructural component (NC), it should have the ability to transfer the inertia load to the main structure, and must be able to accommodate the story drift of the main building. Under the framework of performance-based earthquake engineering (PBEE), reducing the earthquake damage of the NCs, including the CW system, can decrease the cost due to earthquake losses considerably. Accordingly, reducing the earthquake damage to the CW system is a direct objective of the performance-based seismic design.

In this paper, code provisions on seismic demands (i.e., acceleration and drift demands) and seismic experimental methods, damage mechanisms, and performance-based seismic design of CWs are reviewed. The future developments and research tasks of the building CW are also addressed.

## 2. Acceleration demand

Acceleration demand of the CW relates to the seismic design force and corresponding parameters in current code provisions. For a general CW system (e.g., thin glass CW), the calculated seismic force is smaller than the wind load, while for stone cladding, and large glass panel in tall buildings, the seismic force can be larger. Almost in every seismic design provision, seismic design approaches of the general NCs are directly applicable to the CWs. The equivalent inertia force calculation methods are given in many current codes, e.g., Eurocode [49], New Zealand seismic design code [134], British standard [139], ASCE 7–10 [5], Chinese national code [96], Shanghai local seismic code [124], etc. Due to limited space of this paper, seismic provisions of ASCE 7–10 [5] is listed only. The relevant seismic calculation parameters in some widely used codes are reviewed and discussed.

### 2.1. Seismic force calculation

The design provisions in the ASCE 7–10 [5] evolved from the 1994 NEHRP provisions [33]. The main contributor for the new provisions was the Building Seismic Safety Council (BSSC). According to the code, the seismic design forces of the NCs (including CWs) are calculated as follows:

$$0.3S_{DS}I_pW_p \le F_p = \frac{0.4S_{DS}a_p}{R_p/I_p}\left(1 + 2\frac{z}{h}\right)W_p \le 1.6S_{DS}I_pW_p \tag{1}$$

where $F_p$ is the seismic design force applied at the center of gravity of the NC, $I_p$ is the component importance (CI) factor, which takes a value of 1.0 or 1.5, $W_p$ is the component operating weight, $a_p$ is the component acceleration amplification (CAA) factor, which varies from 1.0 to 2.5, the parameter $0.4S_{DS}$ corresponds to the factored mapped design spectral response acceleration at short periods, $z$ is the average height of the NC over the grade, $h$ is the average height of the roof level over the grade, $R_p$ is the component response modification (CRM) factor, it was slightly modified in comparison to the values included in the previous version of the code such that $F_{pNEHRP1994} \approx F_{pNEHRP1997}$ [14]. In the current version of the code, the $R_p$ factor, as defined in section 2.1.2, varies from 1.0 to 5.0. The relationship which involves the $a_p$ term [4,5] was simplified and replaced by a factor $(1+2z/h)$ (floor acceleration amplification (FAA) factor). The adjustment of this factor came from the examination of additional building motion records associated to strong motions with peak ground accelerations greater than 0.1$g$. According to this distribution, the floor accelerations within the building vary linearly from $0.4S_{DS}$ at grade to $1.2S_{DS}$ at the roof level. Additionally, the dependence of the input acceleration on the fundamental period of the primary system, considered in NEHRP 1994 through the structure-response acceleration coefficient $A_s$ [33,4], was removed, according to observations of records obtained for buildings with long natural periods [28,29].

The upper and lower bounds of the seismic design force $F_p$ are intended to ensure a minimum design force, consistent with the values formerly used by practitioners, and considered in the previous versions of the provisions. No recommendations are given for the vertical component of the seismic design force.

### 2.2. Codified acceleration demand parameters

As a NC, the seismic design force of CWs involves the equivalent static force initiated by the floor responses of the main structure. In the past decades, the force based seismic design was very popular in code specifications and guidelines. Due to the fact that acceleration demands are represented by equivalent static forces, researchers paid more attention to the acceleration response for a long time. Even nowadays, the specifications of how to calculate the magnitude of the inertia forces are still the most important part in the seismic design of NCs. These methods are also applicable to the CW system. Most current code provisions involve four parameters, i.e., FAA, CAA, CI, and CRM. Floor response spectrum profile is introduced by AC156 [71] for seismic design and testing of the acceleration sensitive NCs.

#### 2.2.1. Floor acceleration amplification (FAA) factor

In the specified calculation procedure of the equivalent static seismic design force for the NCs, FAA factor is considered in most codes. The FAA factor can be derived from the natural earthquake records or numerical analysis of the structures [47]. The peak value and description of FAA specified in various codes are listed in Table 1. From this table, it is shown that only the New Zealand code [134] provides a bilinear distribution profile, while the other codes provide linear profiles. All the recommended FAA factors reach the peak values on the roof where the amplification effect is the highest. However, in practical engineering design of the CWs, it would be very complicated to use the recommended distribution profiles to calculate the seismic force. Thus, engineers usually use the peak FAA factor. In the design code of China, FAA = 2.0 is proposed in the calculation of the dynamic amplification factor [124].

The roof response will depend on spectral shape of the ground motion and the building's dynamic properties. [102,136,127,128] suggested that, for taller structures, the amplification may vary significantly with height due to higher mode effects. Records obtained during the 1994 Northridge earthquake in multistory buildings showed that floor peak horizontal accelerations were generally greater than those recorded at the ground level [61]. FAA reported by Hall [61] for 25 multistory buildings ranges between 1.1 and 4.6. These results are in

**Table 1**
Floor acceleration amplification (FAA) factor in code provisions [86].

| Code | Description | Peak FAA |
|---|---|---|
| NEHRP1994/UBC1997 | $1+3h_x/h_r$ | 4.0 |
| NEHRP1997/IBC2006/ ASCE/SEI 7-05 | $1+2z/h$ | 3.0 |
| [13] | $K_1=1+2.33z/h$ | 3.33 |
| EC8 (BS EN 1998-1:2004) | $1+z/h$ | 2.0 |
| NZS 1170.5:2004 | $C_{Hi}=1+10h_i/h_n$ for $h_i<0.2\ h_n$<br>$C_{Hi}=3.0$ for $h_i\ge0.2\ h_n$ | 1.0 for $h_i<0.2\ h_n$<br>3.0 for $h_i\ge0.2\ h_n$ [buildings taller than 12.0 m] |
| CSA-S832 2006 | $A_x=1+2h_x/h_n$ | 3.0 |
| GB50011-2010/ J12028–2012 | $\zeta_2=1+z/h$ | 2.0 |
| ASCE/SEI 7-10 | $1+2z/h$ | 3.0 or $a_i$ obtained from response spectra procedure |
| BS ISO 13033-2013 | $1+\alpha z_i/h$ | 3.5 |

**Notes:** $h_x$, $z$, $z_i$, $h_i$ = average height of NC above grade level; $h_r$, $h$ = height of roof above grade level; $h_n$ = height from base of the structure to uppermost seismic weight (mass); $a_i$ = acceleration at the $i$-th level of the structure normalized by peak ground acceleration (PGA); $\alpha\le2.5$ is a parameter that is a function of the type of lateral load resisting system.