McGill University
Department of Civil Engineering and
Applied Mechanics

State-of-the-art review: Seismic response analysis of Operational and Functional Components (OFCs) in buildings

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By
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1- Introduction

A building is composed of two main types of components: structural components (see Figure 1) and non-structural components (NSCs) also called operational and functional components (OFCs) (see Figure 2). OFCs are those components or systems housed or mounted in the buildings which are not part of the main or intended load-resisting system of the structure. Therefore, the building structure is commonly called “primary structure” or “supporting structure” and OFCs are also known by alternative names such as "non-structural elements", "building attachments", "architectural, mechanical, and electrical elements", "secondary systems", and "secondary structural elements".

Figure 1- Structural Components (taken from http://openbuildings.com/buildings/the-yellow-building-profile4695/media)

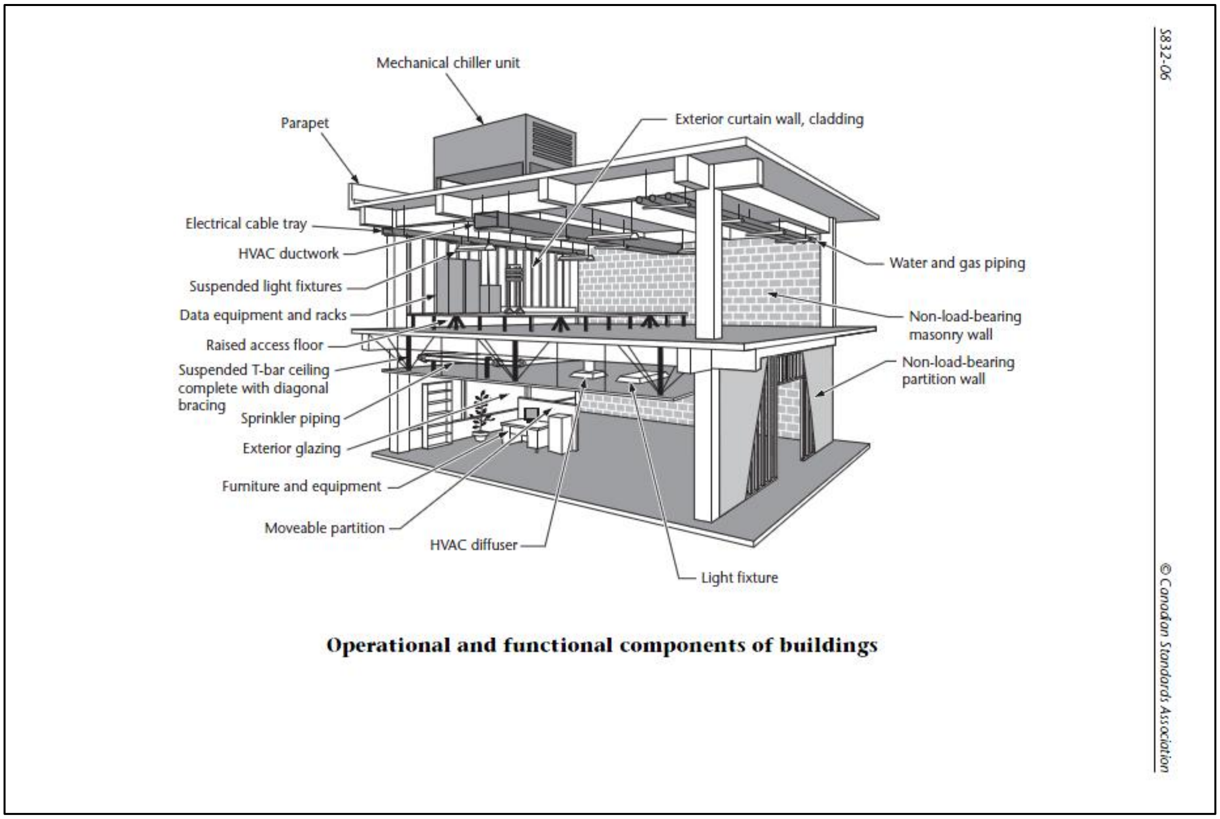


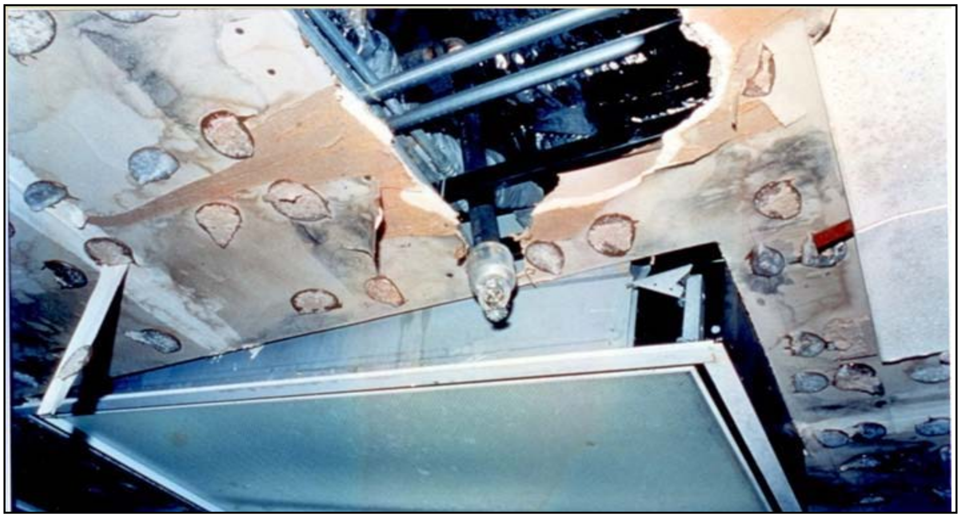
Figure 2: Operational and functional components of buildings (CSA 2006)

According to CSA S832-06(R11), OFCs can be categorized into three sub-groups according to their function: Architectural (external or internal), Building services (mechanical, electrical, and telecommunication), and Building contents (common and specialized) (CSA 2006; Villaverde 2009). They can also be classified into three categories according to the nature of their seismic response sensitivity: 1- Inter-storey-drift-sensitive components, 2- Floor-acceleration-sensitive components, and 3- both Inter-storey-drift- and floor-acceleration-sensitive components. Based on their intrinsic stiffness and the stiffness of their anchoring system to the building structure, they can be grouped as rigid, flexible and hanging type components. A component (considered here with its anchoring system) is defined as rigid if its fundamental sway period is less than or equal to 0.06 sec (frequency above 16 Hz) (Building seismic Safety Council 2003): such components are expected to follow floor/roof building motions without further dynamic amplification. As such, the dynamic properties of rigid components depend primarily on the stiffness of its anchors to the supporting structure. Flexible components are those that have inherent flexibility due to their configuration (pipes, racks, etc.) and/or otherwise rigid components connected with flexible anchors. Such components are prone to dynamic amplification of the floor/roof motions and should be analysed accordingly. Distributed components can be modeled as multi-degree-of-freedom (MDOF) systems or continuous systems with distributed mass and stiffness. They are typically connected by multiple attachments to the buildings (e.g. pipes, cable trays). For the third category of systems hanging from the ceiling (ex. Suspended ceilings, lighting fixtures, other components located in the ceiling plenum) the best way to model them is by single (or distributed) mass pendulum (Sankaranarayanan 2007; Taghavi 2003).

Although OFCs are called secondary systems, they are far from being secondary in importance in terms of functionality and economical value. Their functionality and performance during and after an earthquake is of great significance especially in post-disaster structures such as hospitals, emergency shelters, power stations, etc. As a matter of fact, the good seismic performance of OFCs is essential to achieve the life-safety performance objective that is mandatory for all buildings in Canada (National Research Council Canada (NRC) 2010).

The failure of OFCs during an earthquake can directly threaten the life of building occupants or passersby and impair safe egress procedures. In addition, the failure of some critical OFCs can seriously impair the functionality of post-disaster buildings that should be guaranteed by design. For examples, hospitals should resist design earthquakes without the need for their evacuation. This was an issue with several major hospitals following the 1994 Northridge earthquake in Los Angeles, California (magnitude of 6.7) , which had to be evacuated not because of structural damage but due to (a) the failure of water lines and water supply tank; and (b) the failure of emergency power systems and heating, ventilation, and air-conditioning units (Hall 1994) (See Figure 3).

a)



b)

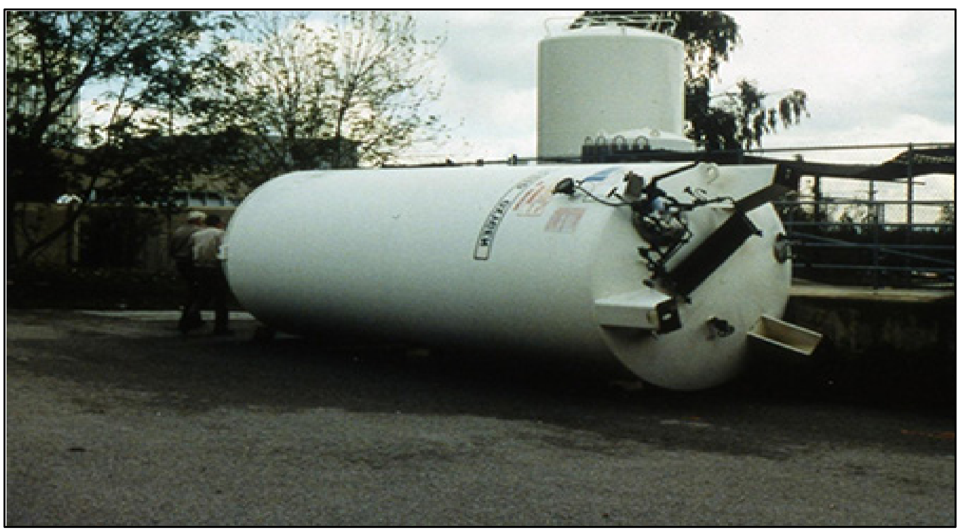


Figure 3: OFC Damage during the 1994 Northridge Earthquake, California: a) Broken sprinkler pipe; b) Vertical tank at hospital overturned due to inadequate anchorage (FEMA E-74 2011).

Life-threatening hazards may result from the collapse of suspended ceiling systems, lighting fixtures, fall of heavy partition walls, collapse of heavy equipment, bookshelves, etc. Exterior components like parapets, signboards, and facade panels may also fall off the building and can cause serious threats to injury or death. An unfortunate example of this type is the death of a student who was struck by a falling precast panel during 1987 Whittier Narrows earthquake with magnitude of 5.9 (Taly 1988)(See Figure 4).

a)

b)

Figure 4: a) Failure of office partitions, ceilings, and light fixtures in the 1994 Northridge Earthquake; b) Failure of precast panel at parking garage that resulted in fatality in the 1987 Whittier, California earthquake (FEMA E-74, 2011).

Lastly, as OFCs represent a large portion of the total cost of the building (e.g. 65 % to 85% of the total cost depending on their use and occupancy), their damage can result in important economic losses (See Figure 5). The financial impact arising from OFCs damage can be divided into direct and indirect economic losses; direct losses are the costs associated with replacing or repairing the failed OFCs, while indirect losses result from business interruption (CSA 2006; Taghavi 2003).

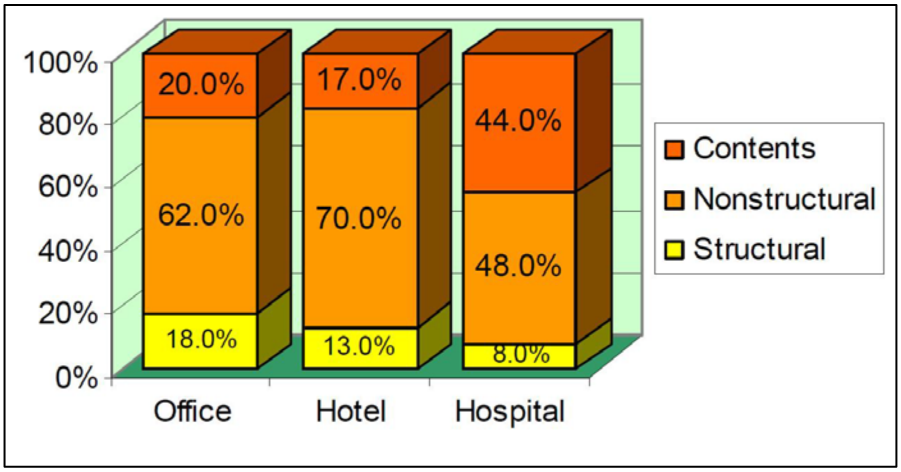


Figure 5: Typical investments in building construction (Soong and Lopez Garcia 2003).

Experience and observations from past earthquakes and current understanding of the seismic behaviour of building structures indicate that OFCs are exposed to large seismic forces during an earthquake and they deserve rational and careful seismic design and analysis procedures of their own,

2- Physical properties of OFCs

OFCs possess several physical characteristic which increase seismic risk and vulnerability associated with them. These characteristics are as follows (Taghavi 2003; Villaverde 2009):

1- In medium- to high-rise buildings, some functional components related to building services are usually located at the higher elevation of the building which makes them exposed to amplified seismic displacements and accelerations compared to ground motion. The amplification of floor accelerations is typically three times the ground acceleration at the upper roof level, and it saturates rapidly above the lower few levels.

2- In general, the stiffness and weight of isolated components are both much lower than those of the supporting structure. As a result, their natural frequencies might be close to one of the natural frequencies of the supporting structure which causes resonant OFC motions.

3- Apart from architectural components, OFCs have typically low damping ratios compared to the building structure. Consequently, they cannot benefit from the fast damping of the effects of strong motions.

4- Architectural components and distributed OFCs are usually multiply-supported, which means that they are attached to the building framework (walls or floors) at different points. Thus, they are subjected to differential motions at their supports and are affected by distortions.

5- OFC supports are mainly designed for purposes other than resisting forces which makes them more vulnerable to even low level seismic motions. This means that damage to non-structural components is normally triggered at levels of deformation and/or acceleration much smaller than those required to initiate structural damage.

3- Important factors in seismic response of OFCs

As mentioned in the preceding section, the physical properties of OFCs make them respond to earthquake ground motions differently from the building structure. Thus, to evaluate the seismic response of OFCs, one needs to account for some parameters that are specifically associated to OFCs. They are including (Chen and Soong 1988; Villaverde 1997):

1- The *dynamic response of the building structure.* As OFCs are attached to or supported by the building, they are directly subjected to the in-building seismic response (floor response) instead of the earthquake ground motion. Such in-building response is typically amplified and filtered according to the dynamic properties of the building lateral load resisting system. 2- *The OFC location along the height of the building*. Owing to different floor responses, two identical components positioned at two different floors in the building will respond differently.

3- *Possible dynamic interaction between OFCs and the building structure.* As mentioned previously, in certain "quasi-resonant" conditions, both the structure and OFCs can interact dynamically and mutually affect or modify each other's seismic response. Well-known rational dynamic analysis techniques are available to consider this effect, where primary (structural) and secondary (OFCs) systems are considered as a coupled system.

4- *Low damping of OFCs*. As mentioned earlier, OFCs normally possess a damping ratio which is much lower than that of the building. This difference in damping ratios of the primary and secondary systems causes the combined system to have non-classical damping and natural frequencies and modes shapes are complex.

5- *Multiple-support excitations.* Multi-supported OFCs are subjected to different and out-of-phase seismic excitations which are exerted at different support locations.

6- *Nonlinear response*. The response of OFCs can be quite affected by the nonlinear behaviour of both the primary and secondary structures.

4- Methods of seismic analysis of OFCs

The seismic response of OFCs is a challenging problem which attracted the attention of many researchers during the past four decades. Attempts have been made to develop rational yet practical methods to analyse the seismic response of OFCs, but researchers have not yet reached a consensus on a generally accepted method to evaluate OFCs' seismic behaviour. This difficulty arises from dynamic characteristics of OFCs that increase the complexity of the problem compared to structural building response such as (Chen and Soong 1988; Singh 1988; Villaverde 1996; Villaverde 2009):

1- *Large number of degrees of freedom (DOFs):* When both the primary and secondary systems are Multi- Degree-Of-Freedom (MDOF) systems, the combined system includes a large number of DOFs which makes the analysis less amenable to simple procedures.

2- *Tuning*: The natural frequencies of OFCs may be close to those of the primary system and this matter causes resonance. Hence, the response of OFCs can be controlled by two or more dominant modes of vibration.

3- *Support configurations*: Multiple supports and various attachment configurations of secondary systems can be quite complicated to analyse (e.g. piping systems).

4- *Non-classical damping*: The presence of non-classical damping in combined systems mandates working with complex natural frequencies and mode shapes and increases the level of complexity of analysis.

5- *Nonlinearity*: The building structure is designed to undergo some inelastic deformations during a severe design earthquake. OFCs themselves might also show some inelastic behaviour in their response which have to be considered as well.

6- *Diversity of OFCs*: There exists a vast variety of OFCs each having different shapes, materials, functions, weight, sensitivity to response parameters of buildings, connections to building, etc.

Despite of all these difficulties, many attempts have been made to develop accurate methods for seismic design and analysis of OFCs and to assure their seismic safety and integrity during earthquakes. These efforts were first initiated by research projects focusing on critical equipments mounted in nuclear power plants such as piping and control systems. In general, the available methods of analysis of OFCs can be categorized into two general groups: 1) Floor Response Spectrum (FRS) approach, and 2) Combined Primary-Secondary (P-S) system approach.

4.1- Floor response spectrum (FRS) approach

4.1.1- Review of early work

One of the first methods developed for analysis of OFCs is the Floor Response Spectrum (FRS) in which the primary and secondary systems are decoupled (i.e. no dynamic interaction between them is considered) and analysed individually. This method is also known by alternative names such as “systems-in-cascade”; or “in-structure response spectrum” (Villaverde 1996). The available technique to determine the FRS can be divided into two general categories: 1- deterministic methods which utilize the time histories compatible with the design response spectra and time-history analysis, and 2- probabilistic methods that use random vibration analysis for determination of FRS from a target power spectral density function (PSDF) without using time history analysis. The latter properly accounts for the uncertainties associated with soil response, materials and inherent uncertainties in seismic motions (Paskalov and Reese 2003).

In the deterministic approach, the response acceleration time history of the primary system at the support locations of OFCs is firstly determined by using the direct time-step integration method given a compatible set of ground accelerograms. This floor acceleration time-history is then utilized as the base-excitation for OFCs to generate a floor response spectrum using either time-domain direct integration analysis or modal superposition (Igusa and Der Kiureghian 1985) (See Figure 6).

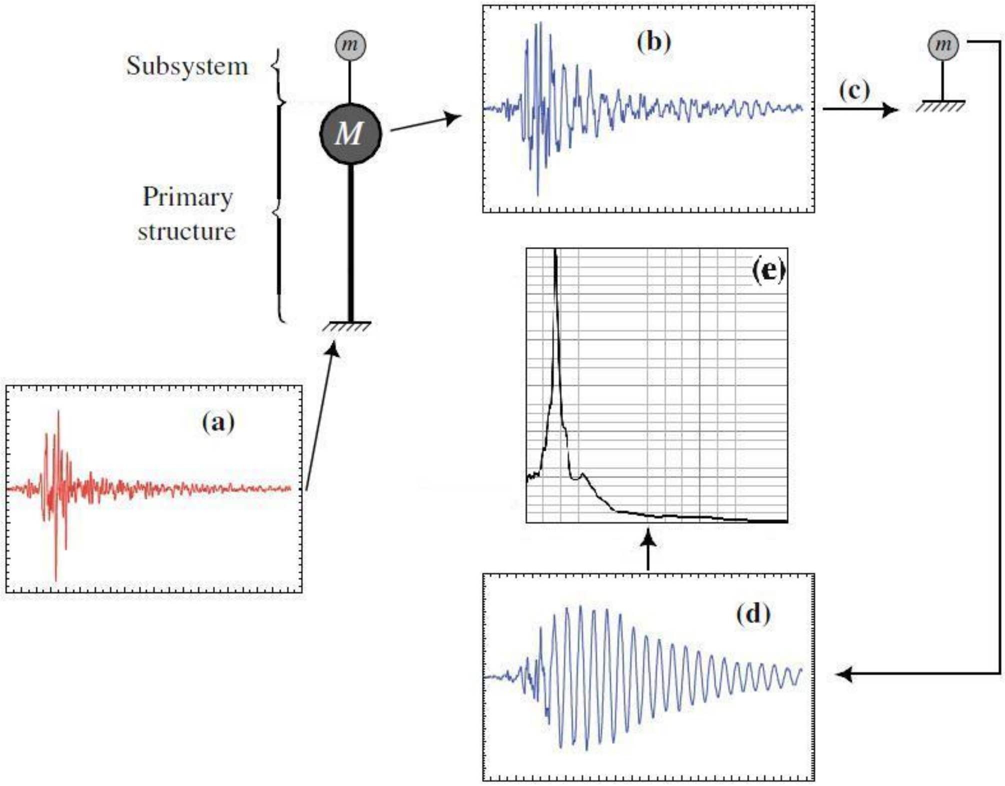


Figure 6: Floor response spectrum approach: a)-Ground acceleration time history as an input for primary structure, b)-Acceleration response-history of the primary structure, c)-Using the acceleration response-history of primary system as the input for secondary system, d)-Acceleration response-history of secondary system, e)-Floor response spectrum

The generated FRS is expected to have peaks at frequencies corresponding to the peaks of the ground motion spectrum and/or at the fundamental dominant natural frequencies of the primary system. For design purposes, FRS peaks are typically broadened to account for the variability in structural frequencies caused by uncertainties in ground motion spectrum, damping, material properties of structure and soil, as well as inaccuracies in the approximation techniques used for modeling and computation in dynamic analysis (Paskalov and Reese 2003). For instance, as described in the USNRC code (U.S. Nuclear Regulatory Commission 1975), in order to determine the amount of peak widening, the sensitivities of structural natural frequencies to each important factor are evaluated first. Then, the expected value of the variation in structural frequency, Δf_j, for each fundamental frequency, f_j, is calculated by taking the square root of the sum of squares (SRSS) of a minimum variation, 0.05 f_j, and the individual frequency variations, Δf_{jn}, as follows:

$$\Delta f_j = \left[(0.05 f_j)^2 + \sum_{n=1}^{p} (\Delta f_{jn})^2\right]^{1/2} \nless 0.1 f_j \qquad (4.1)$$

where Δf_{jn} denotes the variation in the j^{th} mode frequency, f_j, due to variation in parameter number n, and P is the number of significant parameters considered. A value of $0.1f_j$ should be used if the actual computed value of Δf_j is less than $0.1f_j$. If the above procedure is not used, Δf_j should be taken as $0.15f_j$ (U.S. Nuclear Regulatory Commission 1975) (See Figure 7).

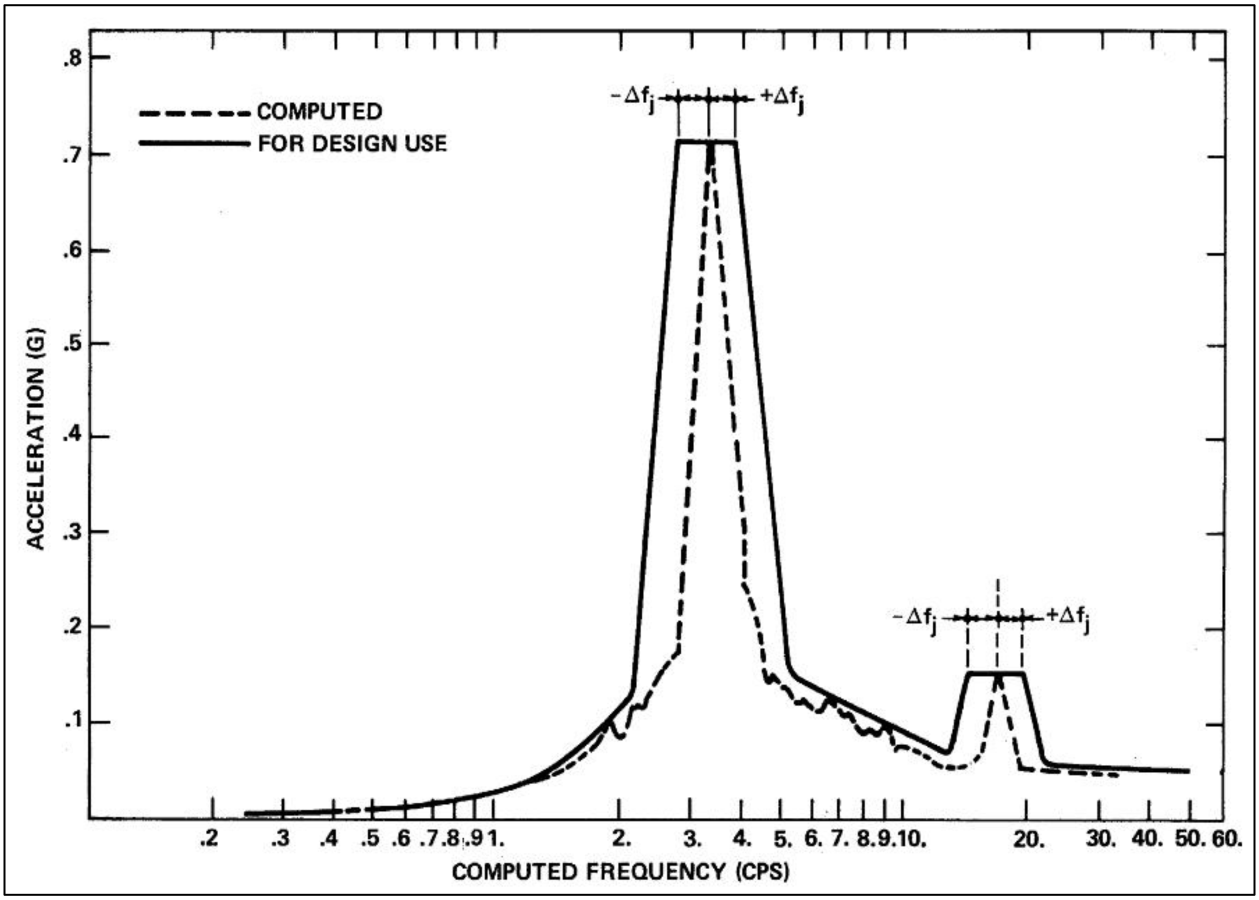


Figure 7: Response spectrum peak broadening and Smoothing (U.S. Nuclear Regulatory Commission 1975)

The response of the primary structure at a OFC support location may have components in three orthogonal directions, which may also come from three-directional excitations (i.e. two horizontal and one vertical in the usual Cartesian system of coordinates). Considering each excitation component, the FRS can be generated at the same location and in the same direction. These individual FRSs can be combined using SSRS technique to derive the total FRS for the given location and given direction (Chen and Soong 1988).

Concerning multi-supported components, an upper-bound envelope of all individual FRSs at support locations can be used to estimate the conservative maximum acceleration response.

Although the method explained above is analytically accurate, the results from a single ground motion time-history are not reliable for design purposes since one ground motion accelerograms cannot represent the characteristics of a possible future earthquake appropriately. So one should consider an ensemble of ground motion inputs and use the average of or envelope to all determined FRSs for OFC design. This series of analytical runs are time-consuming, analytically expensive, and economically unwise. As a result, an alternative approach was introduced to tackle this issue which is called “Spectrum-Consistent Time-History” (SCTH).A spectrum-consistent time-history is an artificially generated ground acceleration time-history whose response spectrum closely envelops the prescribed ground design spectrum and it is used as the excitation input for the primary structure to generate the FRS. Several techniques have been suggested to obtain the SCTH (Iyengar and Rao 1979; Kaul 1978; Preumont 1984; Preumont 1985). However, it was observed that different SCTHs that all envelop the target design spectrum in the same manner, can result in quite different FRSs (Singh and Chu 1976; Singh, et al. 1973), which means that the artificial time-histories are not uniquely defined. Thus, to generate an appropriate FRS for OFC design, one should carry out the analysis for a set of SCTHs and utilize the average of or envelope to all derived FRSs, which is also a time-consuming process.

To overcome these problems and also avoid time-history analysis altogether, great research efforts have been made to develop alternative approaches that can derive the FRS directly from the design spectrum without generating any intermediary input such as the floor response time-history. The result of these efforts is what is named as “Direct methods”. These methods generate the FRS directly based on the design spectrum and the dominant modal properties of the primary structure. These methods are applicable to the linear building structures. Examples are the works done in the 1970s (Amin 1971; Biggs 1971; Biggs and Roesset 1970; Kapur and Shao 1973; Peters, et al. 1977; Singh 1975; Singh 1980), some of which are briefly explained below.

Biggs and Rosset (1990) were among the first to propose the direct method. They suggested the derivation of magnification curves which were obtained from the observed response of secondary systems subjected to a set of recorded seismic ground motions. Their method is semi-empirical and gives conservative results. They divided the equipment into two groups: rigid equipment whose maximum acceleration is the same as that of the supporting point on the structure and very flexible equipments which behave as though supported directly on the ground, as they mentioned. Between these two extreme cases, there exist a wide range of dynamic interactions and resonant effects between the two systems. It is assumed in their study that the structure and equipments will behave elastically. Using the suggested magnification curves, one can simply calculate the maximum modal acceleration response of the equipment directly from the ground motion response spectrum and combining these maximums will give the maximum acceleration response of OFCs.

Singh (1975) also proposed a direct method to obtain the FRS, based on the assumption that the earthquake motions can be modeled as homogeneous stationary Gaussian random processes. Having a Gaussian seismic input, the response of a linear structure will be also Gaussian. Only two factors are required to define a Gaussian process: its mean value and correlation function. Thus, the method is developed to calculate these factors using the power spectral density function (PSDF) of the input ground motion and the dynamic characteristics (lower natural frequencies) of the structure. Knowing these two factors, one can determine the PSDF of the floor acceleration. The variance of the absolute acceleration of the oscillator connected to the floor is determined using another formula developed in the study. Then, the maximum response of the oscillator is equal to the amplified standard deviation by an appropriate factor. Hence, only the dynamic characteristics of structure and the prescribed ground motion are required for this procedure. The main limitation of this method is that the structure should behave linearly. In addition, this approach cannot be used when the OFCs is tuned with one of the fundamental frequency of structure where the FRS usually shows the highest peaks. Supplementary work was done by Singh in 1980 which extends the developed method to the cases in which the OFCs are tuned with the primary structure but still for linear structures only.

Some other direct approaches are based on random vibration analysis in which a MDOF structural system is subjected to a stationary random excitation. Knowing the dynamic properties of primary system, the power spectral density function of structural floor can be directly derived from that of the ground accelerograms. Then this floor power spectral density function is used as input to generate the floor response spectrum. Examples of this method are works by Singh 1975; Vahi 1975; Vanmarke 1977.

Vanmarke (1977) proposed a procedure to obtain the response of a secondary system directly from specified ground response spectra. In his method, the maximum acceleration of a single DOF secondary system is presented as the square root of a sum of contributions which depend on two factors: 1- pseudo-acceleration response spectrum (ground) for the period and damping of the primary system mode k, and 2- the pseudo-acceleration response spectrum (ground) for the equipment period and damping. The Spectral Density Function (SDF) of the absolute acceleration response of the structure at the support point of the secondary system is derived using the dynamic properties of the structure and SDF of ground motion. Then, this absolute acceleration SDF of the primary system is used as input for the random vibration analysis of the secondary system. The SDF of the secondary system response is calculated directly using this input and transfer function/frequency response function of the secondary system. Using the random vibration analysis and SDF of the secondary system response, a formula is suggested to derive the maximum acceleration response of a secondary system directly from specified ground response spectra.

4.1.2- Advantages of FRS approach

The FRS approach is a simple analysis method which allows uncoupling the primary and secondary systems and evaluating their response independently. In comparison with the combined primary-secondary (P-S) system model, FRS method is faster, more economic in terms of analysis time and computational costs. It avoids the numerical complexities that could be encountered in the combined P-S models due the large number of DOFs and considerable differences in terms of the damping ratios, stiffness, and mass of primary and secondary systems. Furthermore, once the floor response spectra are specified, the method then allows the analyst to work on the secondary system independently of the primary system characteristics.

4.1.3- Disadvantages of FRS approach

Despite its simplicity, the FRS method has been proven to be reasonably precise when considering the OFCs that are quite lighter than the primary system and that have natural frequencies not close to those of the supporting structure. When these conditions are not satisfied, however, the FRS method can lead to some gross error or over conservative results in seismic response analysis of OFCs.

As instances, some researchers have recommended that the decoupling the primary and secondary systems is acceptable when the mass of the OFC is less than 1% of the total mass of the supporting structure (Amin 1971; U.S. Nuclear Regulatory Commission 1975). Some shortcomings of the FRS approach which are as follows:

1- As mentioned earlier, no dynamic interaction is considered between the primary and secondary systems in FRS as they are decoupled and analysed independently. When this assumption is not correct, the motion of OFCs may modify the motion of the primary system which in turn affects the response of OFCs (Gupta and Tembulkar 1984). Though neglecting dynamic interaction is usually on the conservative side for acceleration-sensitive components, in some cases it may be grossly conservative and uneconomical (Villaverde 2009).

2- FRS cannot take into account the effect of large differences existing between the damping ratios of OFCs and their primary system (i.e. non-classical damping effects), which makes them vibrate out-of-phase. Non-classical damping effects can be significant when the non-structural to structural mass ratio is small and when the OFC is tuned with the supporting structure (Villaverde 1997).

3- Cross-correlation between the support excitations of multi-connected OFCs is addressed improperly or completely ignored in the FRS method (Wang, et al. 1983). Several empirical techniques have been proposed to account for this problem. As such, Thailer (1976) suggested to obtain the response of the primary structure at different support locations. Then each of these acceleration time-histories are utilized as input for the secondary system to calculate a set of floor response spectra. These FRSs are then combined according to an empirical procedure to estimate the true maximum response of OFC. A common procedure is to pick the largest of the maximum response estimates (i.e. FRS) or to combine them using SRSS. Alternative techniques generate a spectrum enveloping the FRSs corresponding to each support point. However, these methods normally result in overly conservative response predictions for acceleration-sensitive equipment, which is not economically justifiable.

4- It is cumbersome to take into account the torsional response of the structure on the seismic response of OFCs.

5- The other difficulty is to take into consideration the eventual nonlinear response of either or both the primary and secondary structures. In this regard, OFCs with natural frequencies higher than the fundamental natural frequency of the primary structure, generally experience response reductions due to: 1-increased damping of the primary structure when it undergoes inelastic deformations (hysteretic damping) and 2- shift of the fundamental natural frequency of the primary structure away from the natural frequencies of the OFCs. The reason for the shift is the period elongation of the supporting structure caused by inelastic behaviour which decreases the total stiffness of the building. On the other hand, some of the OFCs themselves may have ductile anchors with some post-elastic capacity, which can cause further reductions in their response. One approach to account for nonlinear response is to predict the inelastic response of OFCs from their elastic response using response amplification factors; this technique will be further explained later in section 4.3.

4.2- Combined Primary-Secondary System (CPSS) approach

4.2.1- Review of early work

The aforementioned deficiencies of the FRS method have led to the development of other analysis approaches which can overcome these problems. One solution is to consider the primary and secondary systems together as a coupled system. This is called "Combined Primary-Secondary (CPSS) system approach". In this approach the secondary system is assumed as an integral part of the combined primary-secondary system. Both modal analysis and time history integration can be performed in this approach. Two examples of studies regarding combined P-S systems are described next.

Igusa and Der Kiureghian (1985) have suggested a method for response analysis of multi-supported MDOF secondary systems that is capable of considering the effects of tuning, dynamic interactions, non-classical damping and cross-correlation of support motions. The method proceeds in two steps: 1- determining the modal properties of combined P-S system using the known properties of individual subsystems (i.e. modal synthesis which is discussed in details in section 4.3) and 2- modal superposition analysis of the combined system to obtain the response of the secondary system. Considering that the secondary system is much lighter than the primary system, perturbation theory is used to solve the eigenvalue problem of the combined system in the first step to gain its modal properties, based on the modal properties of the primary system.

Villaverde (1987) also proposed a simplified approximate method to predict the seismic response of a multi-supported MDOF secondary system mounted on a nonlinear primary structure. The procedure first calculates the modal properties of the combined system using the independent dynamic properties of the two systems. Then the maximum response of the equipment is predicted using nonlinear FRS and modal combination techniques. This approach accounts for the interaction and non-classical damping effects completely; however, it is limited in terms of application to the buildings with elastoplastic load-deformation behaviour and also to low-mass components. Villaverde classified the modes of combined systems into two types, resonant and non-resonant. A resonant mode is obtained if the natural frequency of both primary and secondary systems is coinciding. The same method was applied to linear systems in a later study (Villaverde 1991).

4.2.2- Advantages of CPSS approach

Considering the primary and secondary systems as a whole, one can incorporate the following parameters into the analysis:

1- Dynamic interactions between the primary and secondary systems. This effect was first studied by Newmark (1972) who used a modal superposition approach on the combined P-S system.

2- Different values of mass, stiffness, and damping ratio for the primary and secondary systems.

3- Cross-correlation between the motions of various supports of multi-supported OFCs (Asfura and Kiureghian 1986).

4- Non-linearity of the primary and secondary systems.

4.2.3- Disadvantages of CPSS approach

Although the CPSS approach resolves many of the problems associated with the FRS method, establishing a combined P-S system normally results in a coupled system with an excessive number of DOFs, with drastic differences existing between the masses, stiffnesses, and damping ratios of the two systems. These conditions render any conventional methods of analysis costly, imprecise and inefficient.

Also, adopting this method for NCS analysis means that every time a change is made in the OFCs' parameters, the whole coupled structure needs to be reanalysed which is not practical, considering that the design of these two systems is conducted by different teams (structural/mechanical/architectural) and at different times.

4.3- Modal Synthesis (MS) approach

In view of the shortcomings of the FRS approach and the impracticality associated with direct analysis of a combined complex P-S system, several methods have been developed that, while considering the interaction between two systems by analysing them as a coupled system, do not involve the difficulties pertaining to direct dynamic analysis of coupled mechanical systems. One of such methods is "Modal Synthesis approach" (MS) that can be thought as sub-category of the CPSS method. In the MS approach, as it can be inferred from its name, the response of OFCs is determined based on the modal superposition analysis of the combined system. But it is different from CPSS in view of the fact that in the dynamic properties of combined system are determined using those characteristics of its individual components when considered independently and not directly from analysis of the whole system. For instance, if using the conventional response spectrum method in this approach, the different steps involved can be summarised as follows:

1- Determination of ground response spectrum or prescribed design spectrum,

2- Calculation of dynamic properties of combined system – natural frequencies, mode shapes, damping ratios, and participation factors- using the dynamic properties of its individual components,

3- Computation of maximum modal OFC response in terms of the given response spectrum and calculated dynamic properties of combined system,

4- Combination of these maximum modal responses using one of the classical modal combination rules such as SRSS, CQC, etc.

Since in this method the primary and secondary systems are considered as a coupled unit, the deficiencies inherent to the FRS such as neglecting dynamic interactions and variable out-of-phase support motions are eliminated here. Formulating the analysis according to dynamic properties of the independent subsystems can resolve the computational difficulties concerning conventional P-S methods. Furthermore, the need to generate response history of each floor as an intermediary input and also the necessity of reanalysing the structure by every change made in OFCs are not concerns any more. Examples of proposed methods using this MS approach are those by Gupta (1984); Newmark and Villaverde (1980); Newmark (1972); Villaverde (1987); Villaverde (1991). Works by Villaverde are explained earlier in section 4.2. Newmark and Villaverde (1980) proposed a similar approach which is limited to linear elastic primary and secondary systems and also to secondary systems that are connected to the primary system at no more than two points.

As observed in the studies by Aziz and Ghobarah (1988); Sewell, et al. (1989); Singh, et al. (1993), nonlinear behaviour of the primary and/or secondary systems may noticeably affect the force response of the latter. Thus, a simple approximate way to account for this effect is using force response reduction factors to modify the linear response of OFCs in much the same way as is done with ductility ratios for buildings. The essential difference is that for OFCs, the total force reduction factor is equal to the product of the reduction factors of both the primary and secondary systems. Suggested methods to calculate these force reduction factors are, for examples, by Newmark and Hall (1982) and more recently by Miranda and Bertero (1994). It should be noted that in some cases, OFCs might show response amplifications instead of reduction, in terms of response acceleration, which usually occurred when fundamental natural frequency of OFC is tuned with one of the higher natural frequencies of the supporting structure and the OFC is located at lower levels of the building. It is important to mention that despite a reduction in the acceleration response, the displacement response will be increased in presence of non-linear behaviour which can be crucial regarding drift-sensitive components. Several studies have been done to determine the response modification factor and the effect of various parameters on this factor such as the level of inelasticity of the supporting structure, the OFC location in the building, the fundamental period of the component and supporting building, their damping ratios, etc. Examples of these works are those by

Lepage, et al. (2012); Medina, et al. (2006); Sankaranarayanan (2007); Sankaranarayanan and Medina (2007).

Medina et al. (2006) evaluated the dependence of peak component acceleration demand on different parameters such as OFC location and damping ratio, and properties of the primary system including modal periods, height, stiffness distribution, and level of inelasticity in the building. The analytical study covered a variety of stiff and flexible, and elastic and inelastic regular moment-resisting frames subjected to a set of 40 ground motions. Based on the results, some recommendations were made for values of modification factors to obtain the acceleration response of elastic OFCs mounted on inelastic structure, from their response when mounted on elastic structure. Herein, OFCs are represented by linear elastic SDOF systems and no dynamic interactions are considered.

Sankaranarayanan et al. (2007) did a similar study to evaluate the main factors that affect the amplification or decrease of acceleration FRS values caused by inelasticity in the primary structure. Three distinct spectral regions were defined namely long-period, fundamental-period, and short-period regions according to the ratio of T_c/T_s (component period to fundamental period of the building) and the effective acceleration modification factors are defined in each region separately.

Lepage et al. (2012) proposed a simple method for determining the horizontal peak acceleration of OFC in terms of the peak ground acceleration. The results of shake-table tests performed on the floor diaphragms of 30 small-scale reinforced concrete structures have been used to develop the model in which the effect of inelastic response of the supporting structure is taken into account. The method was validated using the data measured in seven instrumented buildings during strong seismic motions and also verified analytically performing non-linear dynamic analysis of 6- and 12-storey reinforced concrete frames subjected to a set of 10 ground motions. The ground motions were scaled to three intensity levels to assess the effect of various level of inelasticity developed in the structure on the response of OFCs.

5- Experimental studies

Beside the numerical studies described above, some experimental studies on OFCs have been performed to qualify equipment, to investigate their seismic response when mounted on the building, and to verify some analytical studies. In general, experimental works can be categorized into two groups of tests. The first group refers to testing of secondary systems mounted on the primary system. This means the experiment is conducted on the integrated combined P-S system (See Figure 8). The second group relates to the testing of individual OFCs to evaluate their dynamic properties and load capacity. A few examples of experimental studies are works done by Craig and Goodno (1981); Kelly and Tsai (1985); Marsantyo, et al. (2000); Schneider, et al. (1982).

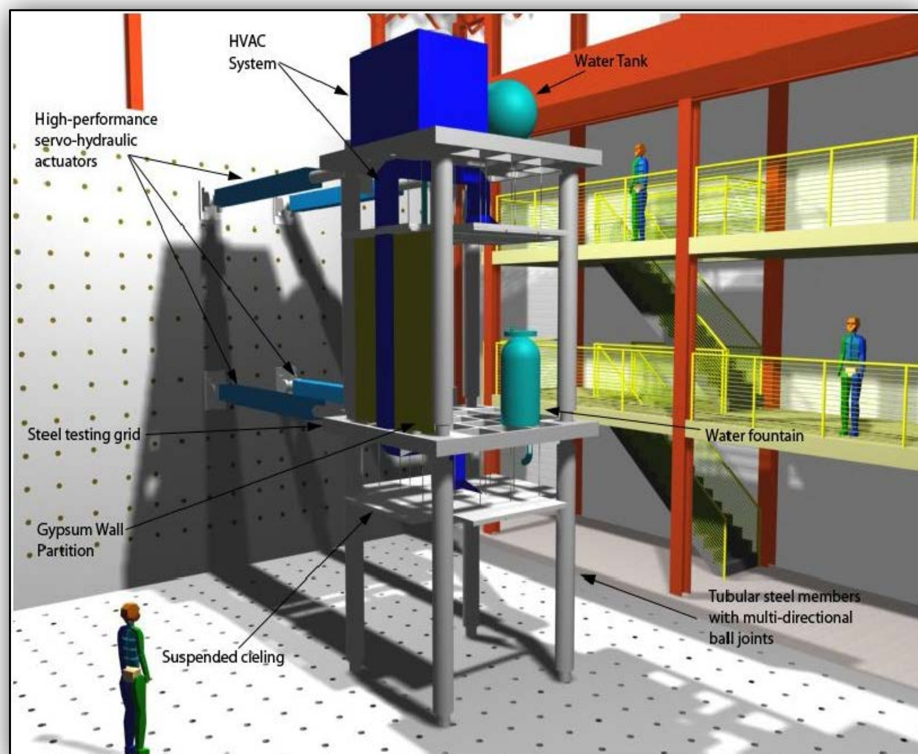


Figure 8: Testing the integrated combined P-S system (Mosqueda. G, et al. 2006)

Craig and Goodno (1981) conducted a series of experiments on full-scale glass cladding panels to measure their natural frequencies, mode shapes, and damping ratios. Their specimens consisted of a single-story section of a cladding system and included the mullions, spandrel framing, glazing materials, and four double-pane vision lights (2.51 x 1.45 x 0.0254 m).

Schneider et al. (1982) performed shake-table tests on one-half scale piping system models typically used in nuclear reactor power plants and mounted on a three-storey steel frame. The experimental investigation addressed both simple and complex piping systems. The piping system was tested in its original design configuration using mechanical shock arrestors (snubbers), and in a revised configuration using ductile steel energy absorbers. The effects of the snubbers and various energy absorbers on the dynamic response of the piping system were studied. The response of the structure was investigated under all three Cartesian components of ground motions. More than 100 tests were conducted in which four artificial earthquakes and sinusoidal excitations were used as inputs. The study addressed the damping behaviour, frequency spectra, and hysteresis loops for both shock arrestors and energy absorbers.

Kelly and Tsai (1985) investigated the response of light equipment in structures isolated using rubber bearings, and compared it with the equipment's response in a fixed-base system. The test setup comprised three oscillators, representing light equipments, attached to the fifth floor of a 1/3 scale five-story frame mounted on four rubber, or lead-rubber, isolators. The total mass of the structure was 36,320 kg. Three isolators were used that weighted 36, 18, and 9 kg. The isolators were tuned to the fundamental natural frequency of the fixed frame, the second natural frequency of the base-isolated frame, and the third natural frequency of the base-isolated frame, respectively. The dynamic response of the equipments was studied in terms of the influence of fixed-base and isolated-base structure.

In the study by Marsantyo (2000), the maximum acceleration amplification factor of OFCs mounted on a building floor was assessed through shake-table tests on two types of acceleration-sensitive components including building equipment and building contents. Four recorded strong earthquake motions were utilized as inputs. Various types of connections of OFCs to the floor were considered. Moreover, the effects of seismic base isolation in reducing the response of OFCs were evaluated.

6- Building code and standards requirements for seismic design of OFCs

6.1- General

Recent building codes address the seismic design of OFCs in new buildings. Some examples in United States are the Uniform Building Code (UBC) (International Conference of Building Officials 1935), the National Earthquake Hazard Reduction Program (NEHRP) provisions (Building Seismic Safety Council 2000), ASCE/SEI 7-10 (American Society of Civil Engineers 2010), the Recommended Lateral Force Requirements and Commentary (Seismology Committee 1990), and the American Society of Mechanical Engineers (ASME) boiler and pressure vessel code (ASME boiler and pressure vessel code 1993). Examples of Canadian codes in this regard are CSAS832-06(R11) (CSA 2006) and the National Building Code of Canada 2010 (National Research Council Canada (NRC) 2010). The older versions of NBC also contained some provisions regarding the seismic design of OFCs in terms of the seismic force and inter-storey drift demand requirements (Assi 2006).

Common limitations which can be pointed out concerning the recommendations of international codes for seismic design of OFCs are: 1- most of them neglect the effect of OFC damping when estimating the acceleration demand, 2- They usually do not consider the effect of higher building modes in their OFC force calculations although this can become important when dealing with high-rise buildings (Sullivan, et al. 2012). Some of these standards provisions for OFC seismic design are discussed below.

6.2- Uniform Building Code

Since its inception in 1935, the UBC (International Conference of Building Officials 1935) of the United States has required the element of structures (e.g. infill walls and etc.), permanent OFCs, and their attachments (e.g. anchors and connections) to be designed for the lateral seismic force, F_p, calculated according to the following formula:

$$F_p = ZI_pC_pW_p \tag{6.1}$$

Where

Z = zone factor representing the expected peak ground acceleration with return period of 475 years.
I_p = Importance factor of OFCs, which is set equal to 1.0 and 1.5 for ordinary and critical components, respectively.
C_p= coefficient specified by the code, having a value ranging from 0.75 to 2.0 depending on the type of component or equipment.
W_p = total weight of the component.

C_p is intended to account for the dynamic amplification of the ground motion by the building for items located above grade. This equation is intended to be used in conjunction working stress design principles which are no longer in use in Canada. The suggested formula by UBC is mainly derived empirically and not based on structural dynamics principles. Hence, it does account for some important factors such as: 1- dynamic interaction; 2- the location of OFCs along the height of structure; 3- attachment configuration and the way the component is connected to the building; 4- tuning or detuning of OFCs with the primary structure ; 5- Cross-correlation and distortion between supports of multi-supported components; 6- Nonlinearity.

6.3- NEHRP Provisions (1994)

Similar to UBC, the United States NEHRP provisions (Building Seismic Safety Council 2000) also provide minimum requirements for seismic design of OFCs and permanent components attached to the building and are intended to use in conjunction with ultimate stress design approach. The requirements are composed of two parts: a minimum required equivalent static force, F_p, and minimum relative displacement demand, D_p, for multiple-supported components. For static force calculations, two formulas are suggested: the first one is conservative and straightforward:

$$F_p = 4.0 C_a I_p W_p \qquad (6.2)$$

The second one is more comprehensive as it includes the effects of more parameters and generally yields lower forces than Equation 6.2.

$$F_p = \frac{a_p A_p I_p W_p}{R_p} > 0.5 C_a I_p W_p \tag{6.3}$$

where

$$A_p = C_a + (A_r - C_a)(\frac{x}{h}) \tag{6.4}$$

and $A_r = (0.2A_s) \leq (4.0C_a)$ (6.5)

The description of the variables of the above formulas is as follows:

F_p = seismic design force applied at the component's center of gravity.

a_p = component amplification factor specified in the provisions according to component type (varies between 1.0 and 2.5).

A_p = acceleration (expressed as a fraction of gravity) at the point of attachment to the structure.

I_p = component importance factor specified in the provisions according to component type (equal to either 1.0 or 1.5).

W_p = component operating weight.

R_p = component response modification factor specified according to component type (varies between 1.5 and 6.0).

C_a = seismic coefficient (expressed as a fraction of gravity) specified for the design of the structure (i.e. effective peak ground acceleration).

A_r = acceleration (expressed as a fraction of gravity) at the structure's roof level.

A_s = structural response acceleration coefficient (i.e. ground response spectrum ordinate), expressed as a fraction of gravity determined from equation:

$$A_s = \frac{1.2\ C_v}{T^{\frac{2}{3}}} \leq 2.5\ C_a \tag{6.6}$$

in which

C_v = velocity-related effective ground acceleration specified for structural design.

T = effective fundamental period of the structure in seconds.

The minimum relative displacement demand for multi-supported components is calculated as the minimum value of the following two recommended equations:

$$D_p = \left(\delta_{xA} - \delta_{yA}\right) \tag{6.7}$$

$$D_p = (X - Y)\,\Delta_{aA}/h_{sx} \tag{6.8}$$

where

D_p = relative seismic displacement between component supports.

δ_{xA}, δ_{yA} , δ_{xB}, δ_{yB} = deflections of building under design forces, multiplied by an amplification factor to account for inelastic deformations, at building levels x, y of buildings A, B.

X, Y = heights above grade of component supports at levels x, y.

Δ_{aA}, Δ_{aB} = allowable story drifts for buildings A, B.

h_{sx} = story height.

Comparing to the UBC recommendations, the NEHRP provisions are much improved as they take into account more effective parameters such as the amplification of ground motion at those points of the structure which are above grade, the location of OFC along the height of building, some dynamic amplification caused by component characteristics, ductility and energy-absorption of OFC, and also the expected performance of the components. However, this method has some limitations as well. As such, it accounts for the response amplification of OFCs using two separate amplification factors (i.e. one related to the structure and another specific to the OFC). Hence, it is not fully accounting for the interaction between the two systems. The implication of different importance factors for the building and the OFC is also not fully justified. The other deficiency relating to this provision is that it requires the satisfaction of both the maximum acceleration and relative displacement demands simultaneously which is overly conservative since indeed they do not happen at the same time: OFCs will typically undergo strong accelerations during the strong motion and large displacements after they have suffered some inelastic damage.

6.4- ASCE/SEI 7-10

In the ASCE 7-10 standards the design seismic force for OFC is defined as:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right) \tag{6.9}$$

F_p = seismic design force.

S_{DS} = spectral acceleration at short period (0.2 s).

A_p = component amplification factor that varies from 1.0 to 2.50.

I_p = component importance factor that can be 1.0 or 1.5 according to the type of OFC.

W_p = component operating weight.

R_p = component response modification factor that varies from 1.0 to 12.

z = height in structure of point of attachment of component with respect to the base.

h = average roof height of structure with respect to the base.

There is also one alternative equation and recommendation for the displacement demand. The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate. Seismic relative displacements, D_{pI}, shall be determined as follows:

$$D_{pI} = D_p I_e \tag{6.10}$$

where

I_e = the seismic importance factor of the building which can be 1.0, 1.25, or 1.5 according to the risk category assigned to the building.

D_p = displacement which is calculated according to two different recommendations explained below:

1- Displacements within structures: having two connection points on the same structure, for example structure A, one at a height h_x and the other at a height h_y. In this case, D_p is calculated according to this equation:

$$D_P = \Delta_{xA} - \Delta_{yA} \leq \frac{(h_x - h_y)\Delta_{aA}}{h_{sx}} \tag{6.11}$$

2- Displacement between structures: having two connection points on separate structures, for example structures A and B, one at height h_x and the other at height h_y, D_p is calculated as follow:

$$\mathrm{D_p} = |\delta_{xA}| + |\delta_{yB}| \leq \frac{h_x \Delta_{aA}}{h_{sx}} + \frac{h_y \Delta_{aB}}{h_{sx}} \tag{6.12}$$

where

D_p = relative seismic displacement that the component must be designed to accommodate

δ_{xA} = deflection at building Level x of Structure A.

δ_{yA} = deflection at building Level y of Structure A.

δ_{yB} = deflection at building Level y of Structure B.

h_x = height of Level x to which upper connection point is attached.

h_y = height of Level y to which lower connection point is attached.

Δ_{aA} = allowable story drift for Structure A as defined in the code.

Δ_{aB} = allowable story drift for Structure B as defined in the code.

h_{sx} = story height used in the definition of the allowable drift.

This ASCE standard indicates that a coupled analysis is not necessary if the OFC mass is less than 1% of the supporting floor mass.

6.5- National Building Code of Canada 2010

The first edition of the NBCC in 1941(National Building Code of Canada 1941) contained seismic provisions in an appendix, based on concepts presented in the 1937 United States Uniform Building Code (International Conference of Building Officials 1935). Specific provisions for seismic design of structural and non-structural components in buildings and essential facilities were first introduced only in the 1953 edition. In all editions of the NBCC, the provisions concerning the OFCs and non-structural components are given in part 4 for structural design and commentary J.

The most recent version is NBCC 2010 in which Clause 4.1.8.18 of NBC Division B Part 4 (National Research Council Canada (NRC) 2010)covers the non-structural elements. It suggests the following equation to calculate the lateral equivalent static force, V_p, for which the components shall be designed:

$$V_p = 0.3 F_a S_a(0.2) I_E S_p W_p \tag{6.13}$$

Where

F_a = acceleration-based site coefficient of the building.

$S_a(0.2)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of 0.2 s.

I_E = importance factor for the building.

$S_p = C_p A_r A_x / R_p$

S_p = seismic amplification factor of the component response; the maximum value of S_p shall be taken as 4.0 and the minimum value of S_p shall be taken as 0.7, where

C_p = seismic coefficient for mechanical/electrical equipment as recommended in code.

A_r = response amplification factor to account for type of attachment of mechanical/electrical equipment as recommended in code.

A_x = amplification factor at level x to account for variation of response of mechanical/electrical equipment with elevation within the building = $(1 + 2h_x / h_n)$.

R_p = element or component response modification factor.

W_p = weight of the component or element.

Regarding the displacement demand, NBCC 2010 stipulates maximum inter-story drifts at any level based on the lateral deflections obtained from linear elastic analysis. These limits are 1% for post-disaster buildings, 2% for schools and 2.5% for all other buildings. The lateral deflections obtained from an elastic analysis should be multiplied by R_dR_o/I_E to give realistic values of anticipated deflections, where R_d is the force overstrength factor and R_o represents the energy dissipation capacity of the element or its connections. And I_E is the importance factor of the building. Further details about the improvement of design provisions for OFCs in Canada can be found in work by (Assi 2006).

6.6- Canadian Standards CSA-S832

CSA-S832-06 (CSA 2006) is the Canadian standards for the "Seismic risk reduction of operational and functional components (OFCs) of buildings". This standard is used in conjunction with NBCC for the calculation of seismic demand parameters of OFCs of new buildings while it contains design provisions and guidelines for the seismic risk assessment and mitigation of OFCs in existing buildings. It recommends two approaches to deal with the seismic design of OFCs. They are:

1- Prescriptive approach: it provides general concepts for design and performance of OFCs and includes the application of typical details, provisions, seismic risk mitigation actions published in industry or manufacturer guidelines that describe the design concepts and construction features required to protect OFCs against seismic hazards. This approach is based on sound engineering standards and practices rather than analysis and calculations.

2- Analytical approach: it requires the seismic design of OFCs against the horizontal and vertical forces, drift ratios, and relative displacement induced by the earthquake. These seismic demand parameters can be calculated using:

a) Simplified approximate approaches based on the equivalent static force analysis method described in NBCC Division B Part 4 Clause 4.1.8.18

b) Rational refined methods which based on engineering analysis, research, and experimentation. These methods duly account for the seismic response of the supporting buildings. They essentially include the methods described previously: floor response spectra, acceleration-time history analysis, elastic/inelastic analysis, and 2-D/3-D frame analysis. Refined methods are mandatory for OFCs with mass greater than 20% of that of the supporting floor (or structural component) or 10% of the total building mass.

7- Research needs and future studies

As mentioned earlier, numerous research projects have been carried out in the past four decades resulting in improved understanding of the seismic behaviour of OFCs, and the development of rational and simplified analysis methods to evaluate OFC response. However, a few subjects related to their seismic response still need further investigation. One area in which more research can be conducted is the effect of building and OFC nonlinearity on the behaviour of OFCs. The second interesting topic for further studies is the effect of the torsional motion of the primary system, particularly in irregular buildings, on the seismic response of its OFCs; this effect can be considerable for those components located in periphery of the structure (for example façade elements, building appendages and parapets, roof equipment, etc.) Another area which has capacity for further studies is the application of base isolation in mitigating the seismic risk and demand on OFCs and also structural control of these components.

Despite the great research effort previously done on this topic and the high level of current understanding of the seismic behaviour of OFCs, and despite the availability of numerous rational methods suggested in the literature to evaluate OFC response, the building codes and standards still do not reflect this level of understanding and do not incorporate the developed techniques. This can be mainly attributed to the fact that these methods are too complicated and cumbersome to be used in the design of ordinary OFCs housed in conventional buildings. Therefore, a great opportunity here is to develop an analysis method that should be rational and precise on the one hand, and simple enough on the other hand, while reflecting the true building characteristics. The author will address this topic in his forthcoming doctoral research proposal.

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