Spider Glass Behaviour Under Seismic Action

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SUMMARY:
This paper presents a study on the structural behaviour of laminated glass panels with point-fixed façade system (spider glass) submitted to seismic action so that it aims to a better understanding of this fixing system behaviour. A study was conducted on a set of laminated glass panels used on an office building in Lisbon. A simplified method was used to obtain the maximum induced force on the façade panels. Subsequently several series of numeric tests were conducted using the accelerations records on a real and artificial earthquake. Time history analyses were preformed to assess the existence of resonant effects on the panel due to the building's natural frequency and the sensitivity to damping.

Keywords: Seismic action, Structural glass, Numerical modelling

1. INTRODUCTION

Glass is a material that has been used in construction industry for centuries mainly for the production of windows, but recent developments in science and technology meant that nowadays glass is used in structural demanding applications like façades, girders and columns.

Due to its characteristics glazing façade solutions aims to gather building's envelope in such way that it has become one of the defining features of the twentieth and twenty first century architecture. Modern glazed façade curtain wall systems include either singular aluminium alloy frame glass curtain walls or frameless glass curtain walls.

During an earthquake, due to both in-plane and out of plane loads, glass breakage and fallout poses serious hazard to pedestrians and occupants with considerable economic losses. For example Sucuoğlu and Vallabhan (1997) refer broken windows as the second most serious non-structural damage related with earthquakes.

Although there is some research on the behaviour of glass panels under out-of-plane loads, e.g., wind loads, the combine effect of both in-plane and out-of plane loads that are applied to the panels during an earthquake seems to be a not very well studied subject.In fact, seismic action brings out specific problems not only to designers but to practitioners as well, mainly, due to the lack of, at least well-known, international rules or regulations about this issue. Design codes tend to limit out-of-plane damage by specifying a seismic static load while damage due to in-plane vibration is controlled by imposing interstorey drift limits to buildings (Sucuoğlu & Vallabhan 1997). The scope of this paper is to present the relevant aspects of the seismic loading in point fixed glass panels. Firstly a simplified method based upon the elastic response spectrum is introduced, then the results of time history dynamic analyses are presented.

2. STATE OF ART

2.1 A review on the structural behaviour of glass and design of glazed structures

From a chemical point of view glass is an inorganic and amorphous non-crystalline material obtain by fusing silica with an oxide and when subjected to loads exhibits a more or less perfect elastic behaviour and brittle failure. The brittle nature of this material is related to the lack of slip planes within the mesh of silicon and oxygen atoms to allow plastic deformations (Overend *et al.* 2007). According to Haldimann *et al* (2008) the theoretical tensile strength of glass may reach roughly 30 GPa, however the actual relevant value for engineering applications is much lower (Hess 2004). The explanation lies on the fact that glass' tensile strength depends very much on the mechanical flaws over the surface which may not be visible to the naked eye. Breakage occurs when the stress at the tip of a surface flaw reaches a critical value. Compressive strength of glass is much higher than the tensile strength, however this is of little interest for most structural applications because failure by buckling is reached long before the compressive strength is exceeded (Haldimann *et al.* 2008).Glass' strength is dependent on the duration and intensity of loading, element's size, residual stresses and environmental conditions. Usually, the higher intensity and load duration with deeper the surface flaws, the lower the ultimate bearing capacity (Haldimann *et al.* 2008).

The current state of art in structural glass design is a process that relies on the combination of empirical rules and recommendations and analytical procedures. Two of the most significant regulations about structural glass are the ASTM E1300 (ASTM 2003) and the prEN13474 (CEN 1999), both are applicable only to glass panels subjected to uniform loads. The design method prescribed by the prEN13474 is the only one that directly compares the internal stresses due to loading with the material strength and enables accounting the non linear behaviour (Martins 2011). Except for the prEN13474 most of the design methodologies fail to achieve the desired accuracy of calculation. If the quantification of the action has an assumed level of certainty, the same cannot be said about the resistance. In fact these methods only lightly address, through empirical safety coefficients, many factors known to influence the response of glass such as the type and duration of loading, the element size and environmental conditions. Especially in these methods the effects of geometric nonlinearity are ignored (Martins 2011).

2.2 Point fixed glazed façade technology

A point fixed glazed façade system (Figure2.1) requires the existence of a hole in the glass panel to perform its support. From the technical point of view this is not the best technique to connect brittle materials such glass due to the stress concentration that is generated in the support region, however this type of technology has been widely used in modern construction mostly due to aesthetic reasons.

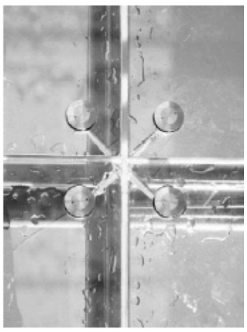

Figure 2.1. Point fixed glazed façade system: Left) example of a point fixed façade; Right) example of a bolt used to perform the panel's support (Martins 2011)

Due to glass' brittle nature, special care should be taken in the design of the supports. Stress distribution around the hole is a complex problem for which there are not design rules. Pilkey (Pilkey 2008) presented some simple formulae, applicable to panels under horizontal shear, that through stress factors allow to assess the maximum stress. Haldimann *et al* (2008) advise the use of the finite element method to determine the final solution, however the designer should be aware the stress distribution is heavily influenced by the modelling solution adopted for the boundary conditions (Martins 2011). In the development of the glass façade the engineer must take in to account some details like the gap between the bolt and the panel, the distance between the hole and the panel's edge and glass' thickness, because this factors are known to heavily influence the panel's structural behaviour (Haldimann *et al.* 2008).

2.3 Seismic design of glazed façades

Severe earthquake causes damage to both structural and non-structural elements such as window glass and curtain walls. Besides the fact that damage suffered by curtain wall façades is very costly, falling façade fragments during an earthquake poses a serious hazard to both pedestrians and people attempting to leave the building.

Very little research has been produced on the seismic performance of glass panels, being one of the firsts due to Bouwkamp (1961); in which the author analysed the behaviour of windows panels under static in-plane loading. Thirty years later King and Lim (1991), as cited by Sucuoğlu and Vallabhan (1997), also published the results of an experimental study on the in-plane behaviour of curtain wall glazing systems with similar conclusions to those of Bouwkamp. More recently, Memari *et al* (2004, 2003) also conducted studies on the seismic behaviour of glazed façade panels.

During an earthquake two types of lateral loads are considered acting in the façade panels: the "in plane" loads and the "out of plane" loads. In-plane actions causes, mainly shear stresses, while the others excite the panel in bending. The frequency content of the dynamic loads transmitted to the panels is modulated by the building natural frequency, so if it happens that it has a value very close to the panel's natural frequency, resonant effects occur with an agonisingly increase of the dynamic response, a well-known phenomena that must be avoided, otherwise structural safety may be compromised.

For out-of-plane loads some simplified methods to assess the seismic behaviour of glazed façade panels are available and will be properly reviewed further in this paper (Camposinhos 2009; Singh *et al.* 2006, 2006), however the same is not true for in-plane loading. As there are no design regulations for determine seismic behaviour under in-plane loads, some laboratory test procedures according to the American Architectural Manufacturers Association recommendations (AAMA 2009, 2009) are followed in order to evaluate the maximum seismic drift which may cause glass breakage and fallout of framed glass panels. The dynamic test procedure considers a sinusoidal drift history with growing amplitudes up to a maximum of 150 millimetres (Figure 2.2). This test method has been applied in previous studies, like Memari *et al* (2003, 2004), and is going to be used to assess the seismic behaviour of point fixed glass panels in the Seismic Laboratory of the University of Porto Engineering faculty

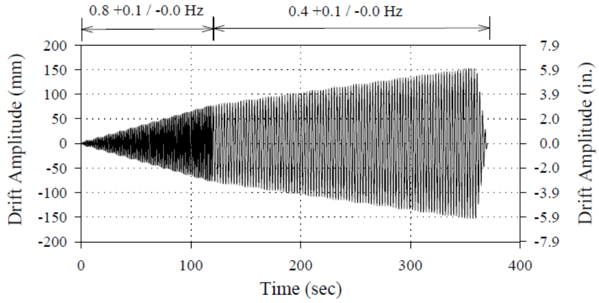

Figure 2.2. Displacement time history for dynamic crescendo test (AAMA 2009)

3. SIMPLIFIED METHOD TO ASSESS SEISMIC LOADS

In the work of Camposinhos (2009) a simplified method to assess the seismic forces, adapted from the studies of Singh *et al* (2006, 2006), is presented. The seismic force, calculated thought Eqn. 3.1, depends on the building's natural vibration period, the panel's mass and natural vibration period and the spectral acceleration evaluated according to Eurocode 8 (CEN 2010) rules.

$$F_{Ek} = \frac{0.40 \cdot C_{fZ} \cdot S_{DS} \cdot \gamma_E \cdot M_E}{R_E} \tag{3.1}$$

Where:

F_{Ek} characteristic seismic force;
C_{fZ} seismic coefficient of the panel
S_{DS} ground acceleration value;
γ_E importance coefficients of the panel (ranging between 1.0 and 1.5);
M_E panel's mass;
R_E coefficient of performance of the panel (ranging between 1.5 and 3.5).

The output of Eqn 3.1 is directly related with C_{fz} which depends on many other parameters like the building's natural vibration frequency and height, the panel's dynamic properties and the floor in which it is located. The individual contribution of each one of these parameters was studied by Martins (2011) in which the author concluded that the positioning of the panel in building plays a major role in the variation of C_{fz} and consequently in the maximum seismic force. The elastic response spectrum used in this study is shown in Figure 3.1. The response spectrum was computed in order to get the maximum spectrum acceleration expected for the Portuguese territory, so was assumed an seismic action type II, soil type D and the Portuguese seismic zone 2.1 (CEN 2010).

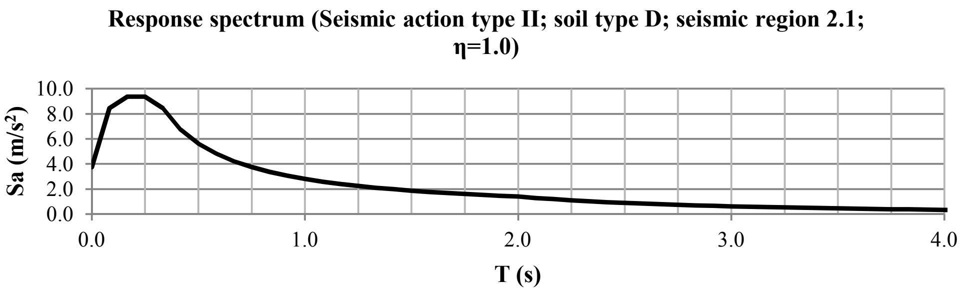

Figure 3.1. Design response spectrum (CEN 2010)

Prior to the application of the simplified method a parametric analysis was made to evaluate the most severe design hypothesis from the seismic action point of view. To evaluate the variation of the parameter C_{fz} two different cases were studied: in the first the panel is assumed to be in the last floor (m=N), and in the second, the panel was assumed to be at the penultimate floor (m=N-1). The results (Figure 3.2) show a significant decay in the parameter C_{fz} when the panel is assumed to be in the penultimate floor with peak values being about 30% lower. Since the seismic load is directly correlated with this parameter the same behaviour is expected, so in the design stage special attention should be devoted to the panel placed in the last floor, thus it is in the significantly severe situation. In the plots is visible that the maximum values are reached for natural vibration periods of the building close the panel's own natural vibration period, this behaviour derives from the fact that the dynamic load transmitted to the panel has its frequency content modulated by the building's natural frequency.

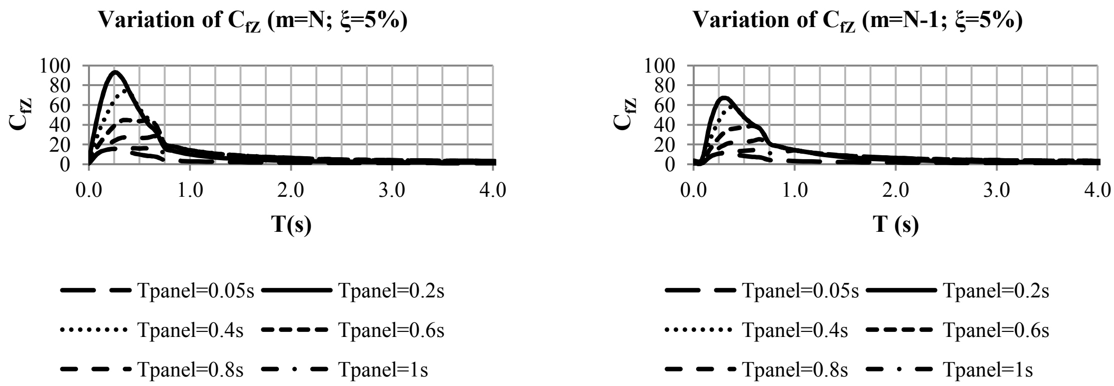

Figure3.2. Variation of C_{fz}: Left) m=N; Right) m=N-1.

4. CASE STUDY

4.1. General description of the test specimens

A set of five point supported laminated glass panels specimens were used to gather the behaviour of a newly built office building in Lisbon. All the specimens have a surface of 2350x2300 mm^2 differing on the glass thicknesses, the interlayer and distance from edges to holes (Figure 4.1).

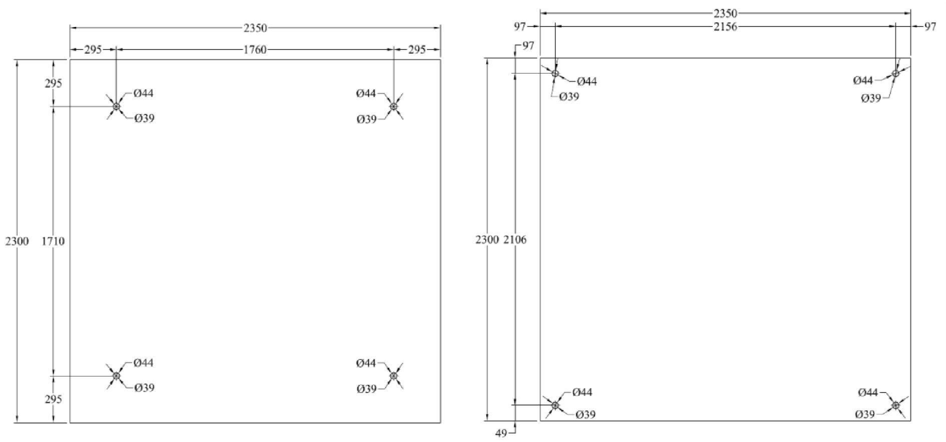

Figure 4.1. Geometrical configuration of the glass panels: Left) V1 and V2; Right) V3, V4 and V5.

Panels identified as V1, V2, V3 and V4 were obtained from 10mm strengthened glass and a 1.52mm thick interlayer together with 8mm tempered glass as well. Panel V5 is made with two 12mm tempered glass sheets. Panels V1 and V3 have a SentryGlass® film while the remaining panels have PVB interlayer. Furthermore an aditional panel with the same geometrical configuration as V5 using SentryGlass® as an interlayer was considered in the analyses. The literature states that PVB's Young modulus ranges from 3.2 MPa to 18 MPa (Chen *et al.* 2010; Delincé *et al.* 2008), so in the panels with this type of interlayer film three different values for the interlayer Young Modulus were considered: (i) 3.2MPa, (ii) 9.0 MPa and (iii) 18.0 MPa. In the case of SentryGlas® a the value of 300 MPa was adopted for its Young modulus, as suggested by Delincé *et al* (2008).Table 4.1 summarises the relevant data for the studied glass panels.

Table 4.1. Test specimen's description.

Glass panel	Dimensions [mm]	Total thickness [mm]	Interlayer	Interlayer's Young modulus [MPa]
V1	2350x2300	10+1.52+8	SentryGlas®	300
V2(i)	2350x2300	10+1.52+8	PVB	3.2
V2(ii)	2350x2300	10+1.52+8	PVB	9.0
V2(iii)	2350x2300	10+1.52+8	PVB	18.0
V3	2350x2300	10+1.52+8	SentryGlas®	300
V4(i)	2350x2300	10+1.52+8	PVB	3.2
V4(ii)	2350x2300	10+1.52+8	PVB	9.0
V4(iii)	2350x2300	10+1.52+8	PVB	18.0
V5(i)	2350x2300	12+1.52+12	PVB	3.2
V5(ii)	2350x2300	12+1.52+12	PVB	9.0
V5(iii)	2350x2300	12+1.52+12	PVB	18.0
V5(iv)	2350x2300	12+1.52+12	SentryGlas®	300

4.2 FEM Model

To evaluate the structural response of the glass panels a set of numerical models were made using commercial finite element (FE) software. The glass panels and interlayer film were modelled with 8-node 3D finite elements. In the interior of the panel the maximum size of the finite elements was limited to 2 centimetres, while near the supports the maximum size was reduced to half to attend the stress concentrations near the holes to take in account the expected stress concentrations in these regions (Figure 4.2).

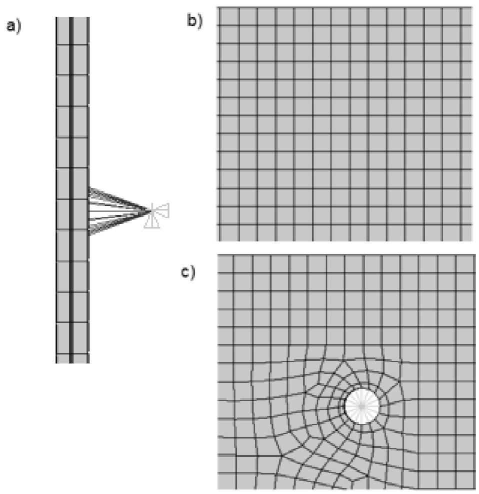

Figure 4.2. FE mesh details - a) lateral view; b) interior of the panel; c) support region.

The degrees of freedom (DOF) restrictions which enabled this stress concentration near the support region and the brittle nature glass leads to a several numerical model simulations stages until a solution that that correctly represent the real support condition was achieved. To assess the behaviour of the model two premises were advanced: (i) the allowance for rotations of the panel according the real behaviour; (ii) Stress distribution around the hole should be compatible with reality. The best solution lead to the implementation of an external node where the support constraints were applied. The connection to the panel was provided by means of rigid beam elements (Figure 4.2).

4.3 Results

4.3.1 Simplified method

After determining the panels natural period of vibration with the FEM models, the already mentioned simplified method was applied to assess the maximum seismic effect on the façade panel. Figure 4.3 relates the expected seismic forces with the building's natural period of vibration.

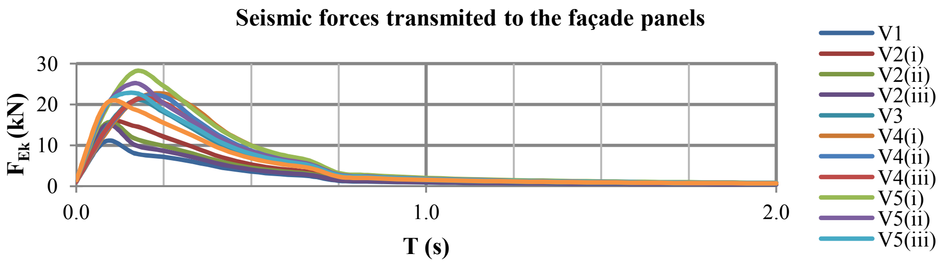

Figure 4.3. Maximum seismic force transmitted to the panels

As it can be observed in Figure 4.3, the maximum values for the seismic forces are all in the order of the tens of kilonewton (nearly 10 times the panel's self weight). These high values lead to the formulation of the hypothesis of being related with resonant effects in the panels due to the frequency content of the dynamic load transmitted to them.

4.3.2 Time history dynamic analyses

In order to qualitatively assess the results obtained by the simplified method a set of time history dynamic analyses has been performed, using the El Centro earthquake ground motion record, properly scaled, and an artificial accelerograms matching the response spectrum in Figure 3.2.

To verify the existence of resonant effects on the panels, a series of dynamic time history analyses were preformed on a simulated building structure carefully chosen so its natural frequency closely matches the one of panel V5(i). The structure's response was later used as dynamic load acting on the panel. The choice for the panel V5(i) for the analysis was based on the fact that the maximum value for the seismic force predicted by the simplified method was calculated for this panel. The building's acceleration and the panel's response is presented in Figure 4.1.To compute the panel's response damping effects in the panel were neglected because the simplified method just considers damping acting in the building structure.

The calculated maximum acceleration response of the panel is greater or equal to ten times the floor maximum acceleration, which confirms the hypothesis of the existence of resonance in the façade panels. As expected the artificial accelerogram imposed to the panel a more severe load than the natural accelerogram, the maximum deflection calculated with this accelerogram more than doubles the one using the El Centro ground motion record. In the same conditions as settled in the simplified method the time history analyses using the artificial ground motion record lead to a similar maximum deflection (Table 4.1), which confirms the suitability of the simplified method to assess the maximum seismic forces acting on façade panel's.

Although the simplified method does not consider any damping acting on the panel, new dynamic time history analyses were preformed to assess the problem's sensitivity to this parameter. Figure 4.5 presents the panel's dynamic response for different dynamic loads and damping coefficients. Comparing Figure 4.5 with Figure 4.5 is clear that damping has a profound effect on the panel's dynamic response. In fact a damping coefficient of 2% leads to decay in the maximum deflection ranging from 30% to 50 %. So in buildings with natural vibration frequencies that might induce resonance in the façade panels an energy dissipation device should be applied to mitigate the effects of dynamic amplification.

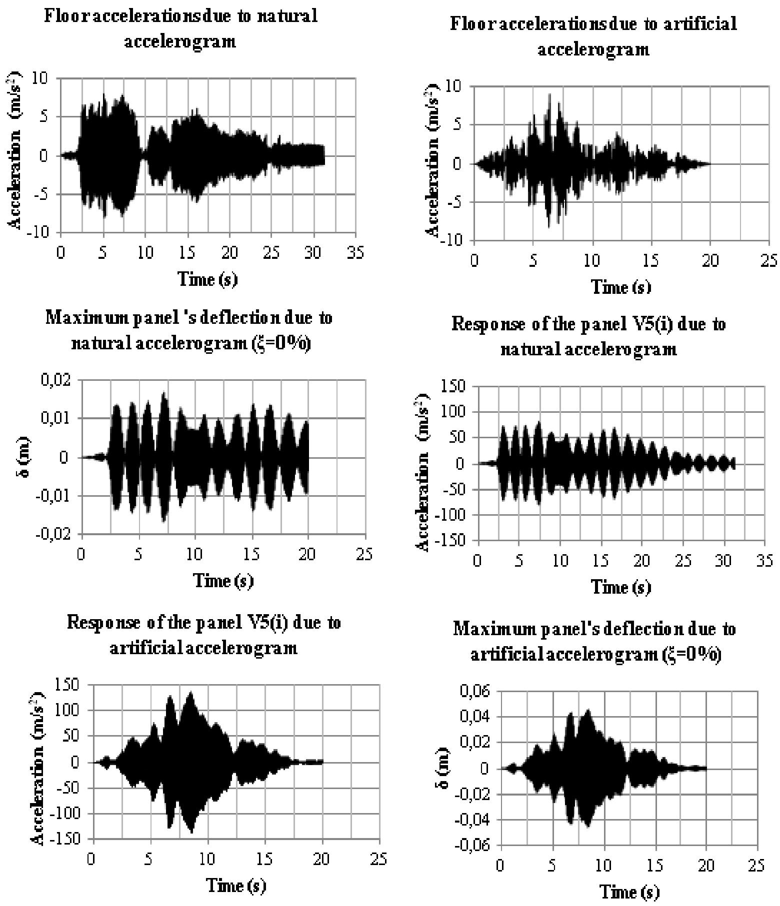

Figure 4.4. Building's acceleration and panels's response

Table 4.1. Comparison between simplified method and time history analyses.

Calculated Maximum Panel Deflection [mm]		
Simplified Method	El Centro Earthquake	Artificial Accelerogram
43.6	16.7	43.0

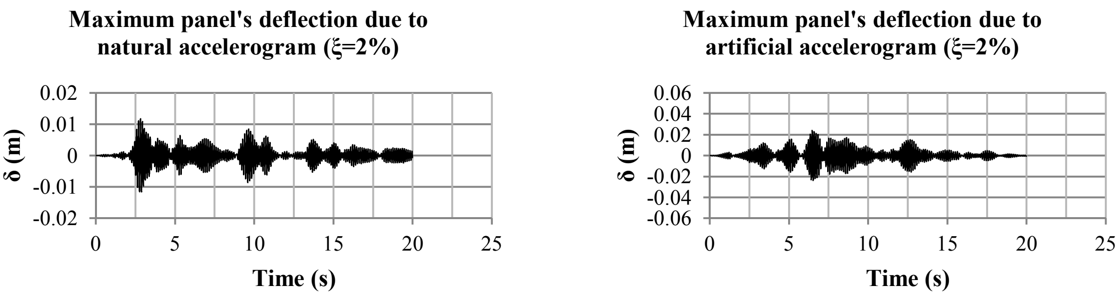

Figure 4.5. Variation of panel's response with dumping.

4.3.3 Numerical simulation of the dynamic crescendo test

In section 2 was briefly described a test procedure to assess the maximum seismic drift causing glass breakage and fallout in framed glass panels. The test procedure, previously described in Section 2 was numerically simulated with the same FE models that were developed to determine the panel's dynamic properties. In Figure 4.6 some results of the calculated maximum stress at different distances from the support region calculated for different drift levels are presented so that a comparison between the maximum stress and the allowable tensile stress was made possible. Thus it can be stated that rupture occurred for drift values less than10 mm.

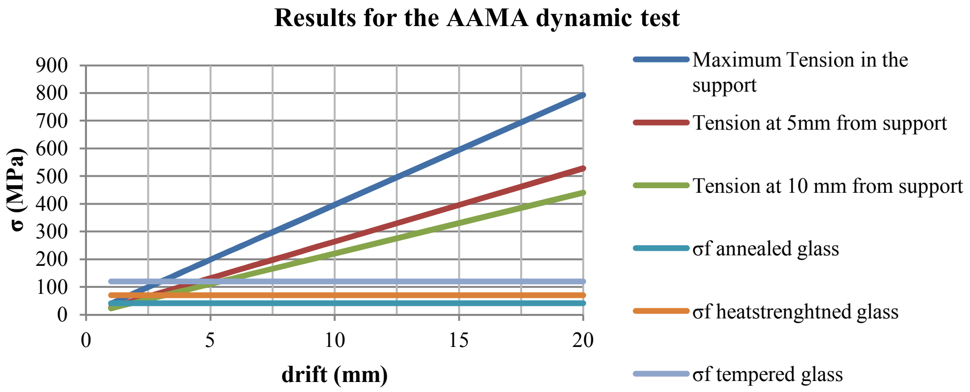

Figure 4.6. An example of the results obtained for the dynamic crescendo test.

Taking as an example the drifts presented in Figure 4.7, calculated for the structure previously mentioned, is clear that the maximum drift that is possible to safely accommodate by the panels is insufficient to ensure safety. So is advisable the application of some special device, such the one proposed by Gowda and Heydari (2010), to ensure that the glass panels can cope with the drift demand. The low drift level that caused material failure determined by the numerical test lead to the formulation of the hypothesis that AAMA 501.6 dynamic test aims to determine the maximum drift that causes panel's detachment from the support rather than the material rupture.

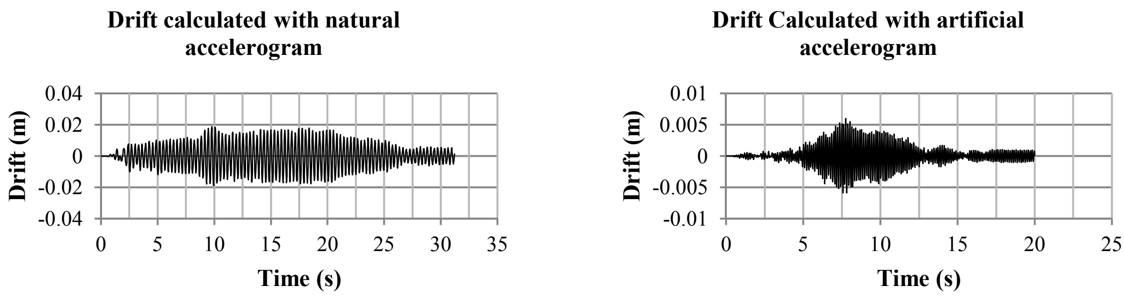

Figure 4.7. Example of drift

5. CONCLUSIONS

In this paper was made clear the importance of an adequate modelling of the support conditions when using FEM to assess the structural behaviour of the panel. The simplified method to determine the seismic forces transmitted to façade panels seems to be able to capture the relevant aspects of the whole issue regarding resonance effects as well. The numerical simulations using time history analysis confirmed the hypothesis of resonance effects induced by the building's natural frequency and are in agreement with the peak values determined by the simplified method. Tests showed up that the problem's sensibility to damping is relevant. In fact a 2% damping ratio for the panels lead to a 30% decrease in its maximum deflection. It must be emphasized that in earthquake prone regions façade panels without energy dissipation devices could be seriously and dangerously excited into non acceptable limits.

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