Transactions on the Built Environment vol 14, © 1995 WIT Press, www.witpress.com, ISSN 1743-3509

# Seismic behavior of nonstructural components: window glass panels

H. Sucuoglu[a], C.V. Girija Vallabhan[b]
[a]*Department of Civil Engineering, Middle East Technical University, Ankara, Turkey*
[b]*Department of Civil Engineering, Texas Technical University, Lubbock, Texas, USA*

## Abstract

Window glass panels respond in two modes during seismic excitation; one "in-plane" and the other "out-of-plane". Simple analytical procedures are developed for calculating out-of plane response of window glass panels mounted inside window frames in relatively rigid structures. The technique is extended to analyse structurally glazed window panels subjected to similar seismic loading

## 1. Introduction

While modern seismic design codes permit some acceptable damage to structural systems which is closely related to their ductility, or deformability under a simulated earthquake, no codes or techniques are enforced to design window glass panels. The damage suffered by the nonstructural elements in modern commercial as well as residential buildings is very much costly when compared with the structural damage (Kripp and Johnson [5]; Arnold [1]; Hirosawa et al. [3]). At the same time, the use of glass panels is very high in modern multistoried and low rise buildings such as shopping malls and such use is increasing at a rapid rate. Window glass panels respond to the absolute floor accelerations transmitted by the structural system in their own vibrational properties when the overall structure is relatively rigid, or they adjust to the dynamic boundary deformations of the relatively flexible structural elements that support them. In either case, there are two modes; one is an "out-of-plane vibration response" and the other is an "in-plane deformation response". If the earthquake is acting at an angle to the building, then the window panels respond in a combination of the two modes. Seismic design codes tend to mitigate nonstructural damage in the out-of-plane vibration mode by designing for equivalent static seismic forces. On the other hand, glass damage due to the in-plane deformation mode is controlled by the story drift requirements imposed on the building structures and the gaps between the glass panels and their supporting frames. A technique is developed here to determine the out-of-plane dynamic response of window glass and structural glazing systems during earthquakes by using simple analytical models. Based on derived expressions, a simple practical procedure for design of glass panels is developed that would sustain the effects of earthquakes. Further research is necessary to develop more accurate and rational design procedures.

## 2. Seismic Response of Window Glass Panels

It is interesting to note that the seismic response of glass panels remain in the elastic range in both in-plane deformation and out-of-plane vibration modes until the panels break. However in the out-of-plane vibration mode of a panel, geometric nonlinearity develops when the amplitude of the maximum lateral displacement becomes larger than half its thickness. These two response modes have to be treated independently as discussed in the following sections.

### 2.1 In-plane Resistance of Glass Panels

Majority of glass failures has been attributed to brittle fracture of window glass panels during in-plane seismic loading. Bouwkamp [2] conducted in-plane loading tests on 39 glazed window panels. Panel size and configuration, type of panel attachment to structural frame, frame material, clearance between glass and window frame, and hardness of putty were the variables in his study. His conclusions were that the in-plane movements of the glazing systems should be accommodated by the rigid body slip and subsequent rotation of the glass panel in the enclosing window frame without stressing the panel. King and Lim [4] also experimentally studied the in-plane behavior of curtain wall glazing systems and reached almost to the same conclusions.

### 2.2 Out-of-Plane Vibration Response and Resistance of Glass Panels

Earthquake damage to window glass panels has not been considered as possible in this response mode. Perhaps, it has been believed that glass panels are flexible enough to vibrate in bending by remaining within the low flexural stress levels. Although this is true for certain cases, many glass failures, especially those on the storefront windows of commercial buildings at the shopping districts during recent U.S. earthquakes are due to excessive out-of-plane vibrations. Since the support conditions of storefront window glasses are different from those of the office windows, their dynamic responses should be evaluated separately.

Transactions on the Built Environment vol 14, © 1995 WIT Press, www.witpress.com, ISSN 1743-3509



### 2.2.1 Storefront Window Glass Panels supported at top and bottom

The glass panel as shown in Fig. 1, resembles a wide, simply supported beam in this configuration. If one assumes that the store frame responds to base excitation of a rigid foundation, presence of comparatively rigid concrete masonry in-fill side walls increases the store frame sway stiffness significantly with identical support motions at the head and the sill. If one examines the earthquake excitation characteristics using the UBC-91 [6] response spectrum for zone 4, soil type two; then identical support motions (at the top and bottom) excite only the symmetrical (odd) vibration modes where the contributions of third and higher modes to the seismic response are negligibly small. Accordingly, the dynamic response of the system can be expressed with its first mode, for which the vibration frequency $\omega$ and mode shape $\phi(x)$ are given by

$$\omega = \pi^2 \frac{t}{h^2}\sqrt{\frac{E}{12\rho}} \quad ; \quad \phi(x) = \sin\frac{\pi}{h}x. \tag{1}$$

Here $E,h,t$ and $\rho$ are the Young's modulus of elasticity, height, thickness and mass density of the glass panel respectively. The maximum out-of-plane deflections and flexural stresses of the glass panels are obtained as functions of thickness and height of the glass panels. The UBC-91[6] design spectrum for type two soil in zone 4 can be reasonably approximated by the following formula

$$S_a = \begin{cases} g, & T < .0575 \text{ sec} \\ \dfrac{0.575g}{T}, & T \geq 0.575 \text{ sec} \end{cases} \tag{2}$$

where $g$ is the acceleration of gravity, and $T$ is the period of vibration. Employing Eqs.(2) in spectral analysis, the following closed form expressions for maximum deflections $W_m$ and maximum bending stress $\sigma_m$ can be obtained as

$$W_m = \begin{cases} \dfrac{48\rho h^4}{\pi^5 E t^2} g, & T < 0.575 \text{ sec} \\ \dfrac{4h^2}{\pi^4 t}\sqrt{\dfrac{\rho}{E}}\, g, & T \geq 0.575 \text{ sec} \end{cases} \qquad \sigma_m = \begin{cases} \dfrac{24h^2}{\pi^3 t}\rho g, & T < 0.575 \text{ sec} \\ \dfrac{4}{2\pi}\sqrt{E\rho}\, g, & T \geq 0.575 \text{ sec} \end{cases} \tag{3}$$

The constant 4 in the second parts of Eqs.(3) has the dimension of time (sec). In the second part of Eq.(3), the maximum stress is found to be independent of $h$ and $t$, as a consequence of the assumption of an inversely varying acceleration design spectrum for the natural periods $T$ >0.575 seconds. Values of $E$ and ρ were used as 70 MPa and 2.6 ton/$m^3$ respectively for the calculation of $W_m$ and $\sigma_m$ that are presented as functions of $h$ and $t$ in Figs. 3 and 4 respectively. Large deflection of glass plates cause shortening in their height, which may lead to a support release at the head, and a consequent failure due to instability as shown in Fig. 1c. The value of this shortening Δ can be computed as a line integral along the deflection profile, as

$$\Delta = \frac{W_m^2 \pi^2}{4h} \tag{4}$$

The values of maximum deflection $W_m$, stress $\sigma_m$ and shortening of height $\Delta$ are evaluated numerically for different heights and thickness of plates with the aid of Eqs.(2), (3) and (4), and the results are presented in Figs. 3, 4 and 5 respectively. These results can be used conveniently for dimensioning earthquake resistant storefront type glass panels (supported at top and bottom) in buildings located in seismic zone 4. It is interesting to note in Fig. 4 that there is an upperbound value of 26.8 MPa (3880 psi.) for maximum flexural stress under the employed earthquake design spectrum.

### 2.2.2 Window Panels Supported with Neoprene Gaskets on Four Sides

A window panel simply supported on four sides, located at an arbitrary story of a multistory building receives uniform absolute accelerations along its edges during dynamic response of the building to a base excitation. This assumption eliminates the contribution of all flexural vibration modes which do not possess double symmetry, leaving the first mode as the only one contributing to the out-of-plane vibration response. Hence the behavior of the glass panel is reduced to that of an equivalent single degree of freedom system with the natural frequency and vibration shape defined as

$$\omega = \pi^2\left[\frac{1}{a^2} + \frac{1}{b^2}\right]\sqrt{\frac{D}{\rho t}}\,; \qquad \phi(x,y) = \sin\frac{\pi x}{a}\sin\frac{\pi y}{b} \tag{5}$$

where $a$ and $b$ are the glass panel dimensions, $D = Et^3/12(1-\mu^2)$ is the flexural rigidity, $t$ is the thickness, $\mu$ is the Poisson's ratio, and $\rho$ is the mass per unit volume of glass. If the absolute floor acceleration response spectra for the boundary excitation is defined as $S_{af}$, then by carrying out the spectral analysis procedure for the single degree of freedom system, the maximum lateral deflection $W_m$ and inertial force $F_p$ of a glass panel is obtained as

$$W_m = \frac{16}{\pi^2}\frac{S_{af}}{\omega^2}\,, \qquad F_p = \frac{64M}{\pi^4}S_{af} \tag{6}$$

where $M$ is the total mass of the glass panel. Definition of a flat floor acceleration response spectrum brings further simplicity into the analysis, and provides a means of comparison with the previously discussed UBC-91[6] and Japanese code requirements. If the floor acceleration spectrum is estimated as $S_{af} = 2g$, then the seismic force coefficient is calculated as 1.314. This value is very close to the Japanese code requirement and 64% higher than the UBC requirement for nonstructural elements in zone 4. A flat floor response spectrum assumption is justified here because Eq(5) yields vibration periods which are considerably shorter than the natural vibration periods of multistory buildings. In such instances dynamic interaction between the glass panels and the building can be neglected.

The effect of geometric nonlinearity is to reduce the natural vibration period and the maximum lateral deflection of glass panels during their seismic out-of-plane response. Vallabhan et.al.[7] using iterative numerical procedures developed solutions for the forced flexural vibration of rectangular glass plates subjected to dynamic pressures. Here their results for different aspect ratios of vibrating window glass plates are converted into normalized relations between natural vibration period and maximum lateral deflection, and maximum flexural stress and maximum lateral deflection as shown in Figs. 6 and 7. A simple iterative procedure is developed here for calculating the maximum lateral deflection and maximum flexural stress of the glass panels with nonlinear flexural response. The results for square glass plate windows of different heights and thickness are presented in Fig. 8. This figure reveals that the stiffening effect of geometric nonlinear behavior reduces the lateral plate deflection, and accordingly the flexural stresses significantly. The resulting maximum stresses under the conservatively estimated floor excitation are fairly low and they indicate considerable seismic safety for window glasses in the flexural vibration mode.

Glass units in structural glazing systems supported with silicone sealant on all sides as shown in Fig. 2 can be considered as plates simply supported on elastic springs along their edges. For the first mode of vibration, using the Rayleigh method, the vibration frequency is obtained

$$\omega = \pi^2 A_2 \left[ \frac{D k_s A_1}{\rho t} \left( \frac{32 A_0 A_2^2 D + A_1 k_s}{16\pi^4 A_0^{\,2} A_2^4 D^2 + 64 A_0 A_1 A_2^2 k_s D + A_1^2 k_s^2} \right) \right]^{1/2} \tag{7}$$

where $A_0 = ab$, $A_1 = (a+b)$, $A_2 = (a^{-2} + b^{-2})$ and $k_s$ is the stiffness of the silicone sealant per unit length, vertical to the plane of the glass unit. Equation (7) reduces to $\omega$ in Eq.(5) as $k_s$ approaches to infinity. The $k_s$ values in practice vary between 1 and 2 MPa. Substituting values of $k_s$ in this range into Eq.(12), it is found that the differences for $\omega$ are less than 10% with respect to the simply supported glass units with rigid supports. Thus it may be concluded that the stiffness of silicon sealants does not effect the flexural dynamic response of glass units in structural glazing systems.

## 3. Conclusions

The findings of this investigation can be summarized as follows:

1. The studies in the earthquake engineering literature on window glass panels are not sufficient to yield a sound design practice.
2. Proposing simple analytical tools for the seismic design of glass components is necessary for mitigating earthquake damage to window glass panels.
3. In-plane deformation capacity of window panels depends on the clearance between glass panel and window frame, and resiliency of the sealant material.
4. Seismic behavior of large storefront glass panels can be analyzed as discussed here.

5. In multistory office buildings which are relatively rigid, window glass panels that are supported on all four edges have sufficient flexural stiffness and strength in the out-of-plane vibration mode against floor accelerations normal to the plane of the glass imposed along their edges.
6. The stiffness of structural sealants in structural glazing systems have negligible effect on the dynamic flexural response of the glass units.

**Acknowledgment**

This work was performed as a part of the research project on the behavior of window glass panels during earthquakes sponsored by the National Science Foundation/ Science in Developing Countries Program under Grant No. INT-9214251. The support of Dr. Edward Murdy, program manager at NSF is gratefully acknowledged

**References**

1. Arnold, C.(1991), "The Seismic Response of Nonstructural Elements in Buildings", *Proc. Pacific Conference on Earthquake Eng.,* New Zealand, Natl. Soc. for Earthquake Eng., Auckland, Vol. 2, pp. 335-350.
2. Bouwkamp, J.G. (1960), "Behavior of Window Panels Under In-Plane Forces", *Structures and Material Research Series 100, Issues 7 and 8*, University of California, Berkeley.
3. Hirosawa, M. Mizuno, H. and Midorikawa, M. (1992), "State-of-the-Art Report on Seismic Design of Building Equipment and Nonstructural Components in Japan", *BRI Research Paper No. 137*, Building Research Institute, Ministry of Construction, Japan.
4. King, A. B. and Lim, K.Y.S.(1991), "The Behavior of External GlazingSystems Under Seismic In-Plane Rocking", *Proc. Pacific Conference on Earthquake Eng.*, New Zealand, Natl. Soc. for Earthquake Eng., Auckland, Vol. 2, pp.315-326.
5. Kripp, J.G. and Johnson, M.W., (1990), "Seismic Loss Estimation for Non-structural Components in High Rise Buildings", *Proc. 4th U.S. National Conference on Earthquake Engineering*, EERI, Oakland, California.
6. "Uniform Building Code-1991 Edition" (1991), International Conference of Building Officials, Whittier, California.
7. Vallabhan, C.V.G., Das,Y.C. and Kamineni, S. (1990), "Nonlinear Dynamic Response of Window Glass Units", *Journal of Wind Eng. and Ind. Aerodynamics*, 36, pp. 1155-1162.

Transactions on the Built Environment vol 14, © 1995 WIT Press, www.witpress.com, ISSN 1743-3509



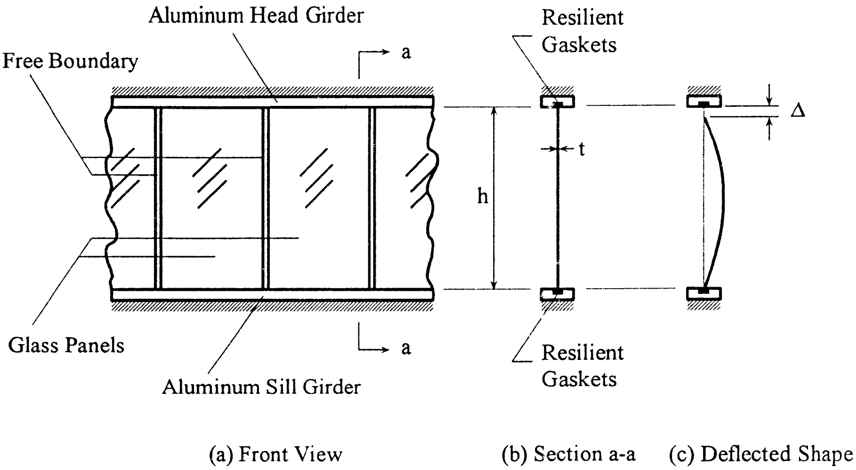


Fig. 1. Connections between Storefront Glass Panels and Store Frames

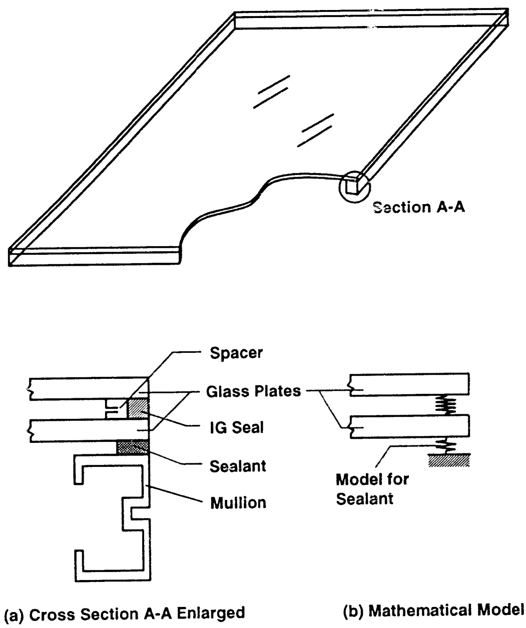


Fig. 2. A Typical Structural Glazing System

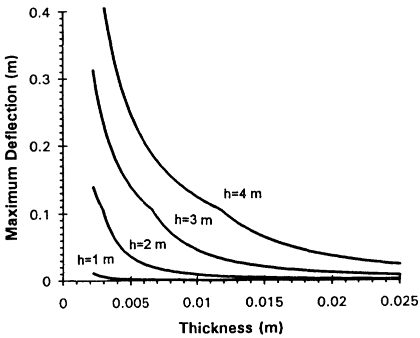


Fig. 3. Max Deflection vs Height and Thickness

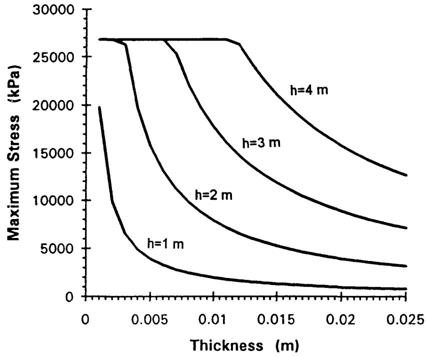


Fig. 4. Max Stress vs Height and Thickness

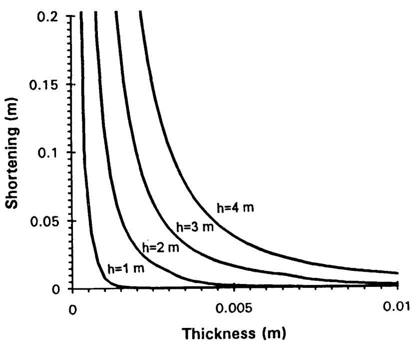


Fig. 5. Height Shortening vs Height and Thickness

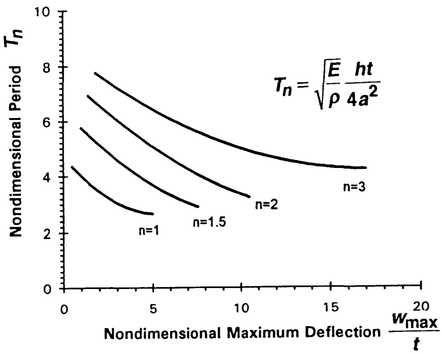


Fig. 6. Period vs Max Deflection

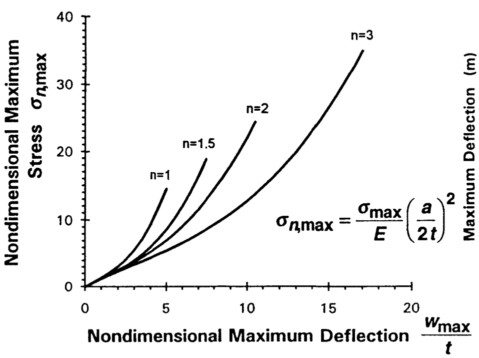


Fig. 7. Max. Stress vs Max Deflection

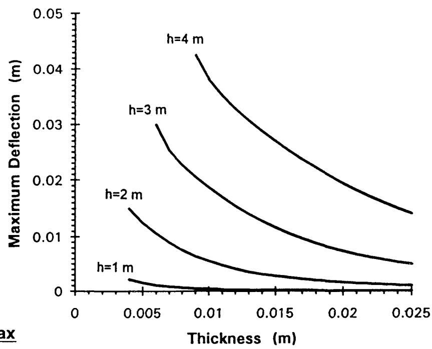


Fig. 8. Max. Deflection vs Thickness of a Square Plate in Zone 4

Transactions on the Built Environment vol 14, © 1995 WIT Press, www.witpress.com, ISSN 1743-3509