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SEISMIC RESPONSE OF RAIL-COUNTERWEIGHT SYSTEMS OF ELEVATORS

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ABSTRACT

This paper presents a methodology to calculate the response of the counterweight of elevators in buildings subjected to earthquake motions. The seismic input to the counterweight is the absolute acceleration time history at the supports of the guide rail. The effect of the torsional irregularities in a building are taken into account in the definition of the seismic input. A three degree of freedom model of the rail and counterweight system is developed. The deformation of the guide rail and roller guides are calculated by means of relative displacement floor response spectra. The floor spectra are obtained by time history analysis and by a direct method.

KEYWORDS

Non-structural components, elevators, counterweights, secondary systems, mechanical equipment, special structures, floor response spectrum.

INTRODUCTION

Following the extensive damage to elevator systems during the 1971 San Fernando earthquake, the elevator industry and the State of California prepared stringent specifications for new elevators and for retrofitting of existing ones. They included, among other measures, the installation of seismic switches and derailment sensors, and the use of larger counterweight guide rails and intermediate brackets. However, during the 1987 Whittier Narrows, 1989 Loma Prieta, and the 1993 Northridge earthquakes, 91, 249 and 690 respective occurrences of counterweights (CW) coming out of their guide rails were reported (Swerrie, 1991; Schiff 1987). Usually, the measures taken to enhance elevator safety are based on experience with previous designs, visual inspections and surveys of elevator companies. There is still a need for more analysis in which to base these recommendations. Moreover, any procedure for the design and verification of the seismic performance of these systems must be simple enough to be used by practitioners who are not structural dynamics experts.

The fact that the counterweight is the most vulnerable part in the elevators was recognized by a few researchers that studied the seismic response of rail-counterweight systems in the plane of the CW. Yang *et al.* (1983) constructed a small scale physical model of the CW system. They compared the experimental results with a mathematical model in which the rail and CW were assumed to be connected by bilinear springs. These studies were continued by Tzou and Schiff. In a series of papers (Tzou and Schiff, 1984, 1987, 1988, 1989; Tzou 1985) they studied the hammering of the CW against the rail. Some measures to reduce the rail loads, such as introducing large gaps between the CW weights and frame and the use of rubber dampers and intermediate tie brackets were also examined . In all these studies the seismic input consisted of sinusoidal forcing functions. Earthquakes records were used by Segal *et al.* (1994). They also used a nonlinear contact element formed by a spring, a viscous and friction damper to tie the CW to the structure.

This paper presents some of the results of an ongoing research project to establish guidelines for the design and verification of counterweight and guide rail systems subjected to earthquake loads (Suarez and Singh, 1995). The elevator is assumed to be located in a building with torsional eccentricities. The building model has three degrees of freedom (dof) per floor: two perpendicular translations of the center of mass and a rotation about the vertical axis. The building base is subjected to one or two perpendicular components of an artificial or recorded earthquake. The absolute acceleration at the supports of the guide rail is then calculated and subsequently used as input for the counterweight analysis. The multiple support excitation induced by real or simulated earthquake motions permits to obtain more realistic results than the in-phase sinusoidal support excitation used in previous studies.

THE COUNTERWEIGHT MODEL

Several simplified models were used to study the effect of the earthquake motions on the counterweight and guiderail system (Fig. 1). Models A, B and C have two degrees of freedom: (u_c, ϕ), (v_c, ψ), and (u_c, θ), respectively. Model D is a combination of models A and C.

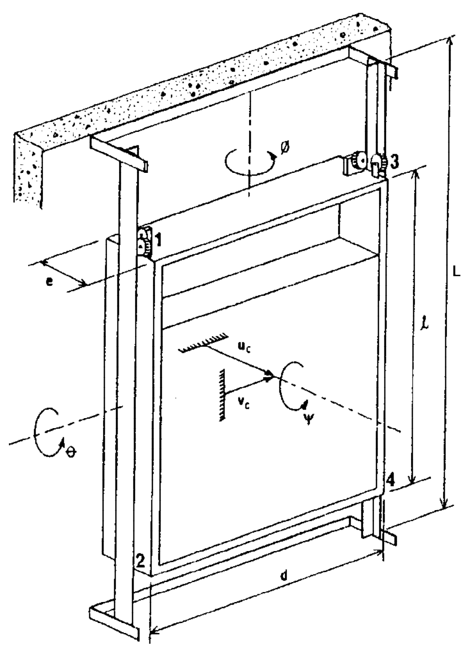

Fig. 1. Guide rail and counterweight system

The following simplifying assumptions were used in the development of the models. The brackets that supports the guide rail are fastened to the building at the level of the floor slabs. The seismic input acting on the counterweight is the acceleration transmitted through the hoistway and guide rail to the rollers. It is also assumed that the roller guides experience the same acceleration than the supports of the rail. The accuracy of this assumption was verified using more elaborate models that include flexible guide rails. The flexibility of the brackets that support the guide rails is neglected in models A through D. The action of the main and compensating ropes attached to the CW frame are not considered. They were included in one of the models but their effect on the dynamic behavior was found to be insignificant.

Only some salient features of the study will be presented in the paper. The reader interested in more details is referred to the report by Suarez and Singh (1996).

The mass of the counterweight, including the mass of the weights and frame, is m. The counterweight has a height ℓ, width d and depth e (see Fig.1). To account for the flexibility of the guide rails and roller guides, they are represented as series springs with total stiffness k_1 and k_2. The coefficients $k_1$and k_2 represent, respectively, the stiffness coefficients at the upper and lower supports of the CW and are defined later.

The equations of motion for model D are

$$\begin{bmatrix} m & 0 & 0 \\ 0 & J_\Phi & 0 \\ 0 & 0 & J_\Theta \end{bmatrix} \begin{Bmatrix} \ddot{u}_c \\ \ddot{\phi} \\ \ddot{\theta} \end{Bmatrix} + \begin{bmatrix} 2(k_1+k_2) & 0 & \ell(k_2-k_1) \\ 0 & \frac{d^2}{2}(k_1+k_2) & 0 \\ \ell(k_2-k_1) & 0 & \frac{\ell^2}{2}(k_1+k_2) \end{bmatrix} \begin{Bmatrix} u_c \\ \phi \\ \theta \end{Bmatrix} = - \begin{Bmatrix} m\ddot{x}_c \\ J_\Phi \ddot{\alpha} \\ J_\Theta \ddot{\beta} \end{Bmatrix} \tag{1}$$

where the excitation terms are defined in terms of the acceleration at the four supports

$$\begin{aligned} \ddot{x}_c &= (\ddot{x}_1 + \ddot{x}_2 + \ddot{x}_3 + \ddot{x}_4)\ /\ 4 \\ \ddot{\alpha} &= (\ddot{x}_1 - \ddot{x}_3)\ /\ d = (\ddot{x}_2 - \ddot{x}_4)\ /\ d \\ \ddot{\beta} &= (\ddot{x}_2 - \ddot{x}_1)\ /\ \ell = (\ddot{x}_4 - \ddot{x}_3)\ /\ \ell \end{aligned} \tag{2}$$

Note that the assumption that the CW is rigid requires that $\ddot{x}_1 - \ddot{x}_3 = \ddot{x}_2 - \ddot{x}_4$ and $\ddot{x}_2 - \ddot{x}_1 = \ddot{x}_4 - \ddot{x}_3$ .The terms J_Φ and J_Θ are polar mass moments of inertia of the CW defined as

$$\begin{aligned} J_\Phi &= \frac{md^2}{12}(1+\gamma_1^2) \quad ; \quad \gamma_1 = \frac{w}{d} \\ J_\Theta &= \frac{m\ell^2}{12}(1+\gamma_2^2) \quad ; \quad \gamma_2 = \frac{w}{\ell} \end{aligned} \tag{3}$$

When $k_1 = k_2 = k_t$, the equations of motion become uncoupled. Since one is usually interested in calculating the deformation of the rails and rollers, it is convenient to use as variables

$$u_e = \frac{d}{2}\phi \quad ; \quad u_e' = \frac{\ell}{2}\theta \tag{4}$$

In terms if these variables, the uncoupled equations of motion become

$$\begin{aligned} \ddot{u}_c + 2\zeta_o\omega_o\dot{u}_c + \omega_o^2 u_c &= -\ddot{x}_c \\ \ddot{u}_e + 2\zeta_o\Omega_o\dot{u}_e + \Omega_n^2 u_e &= -\ddot{x}_{dif} \\ \ddot{u}_e' + 2\zeta_o\Omega_o'\dot{u}_e' + \Omega_o'^2 u_e' &= -\ddot{x}_{dif}' \end{aligned} \tag{5}$$

Note that a damping term was introduced in each of the equations. It can be shown that the excitation terms can be written as

$$\begin{aligned} \ddot{x}_c &= \frac{1}{2}[\ddot{x}_{n+1} + \ddot{x}_n - \frac{y_c}{2}(\ddot{\theta}_{n+1} + \ddot{\theta}_n) \\ \ddot{x}_{dif} &= -\frac{1}{2}\ d\ \ddot{\theta}_{n+1} \\ \ddot{x}_{dif}' &= \frac{1}{2}[\ddot{x}_n - \ddot{x}_{n+1} - \frac{1}{2}(y_c - \frac{d}{2})(\ddot{\theta}_n - \ddot{\theta}_{n+1})] \end{aligned} \tag{6}$$

The inputs to the three systems are defined in terms of the absolute acceleration of the two consecutive floors, $\ddot{x}_n$ and $\ddot{x}_{n+1}$, the rotation of the floors $\ddot{\theta}_n$ and θ_{n+1}, the distance y_c between the mass centers of the counterweight and floor slab measured perpendicular to the direction of the ground motion, and the width d of the counterweight frame (See Fig. 2).

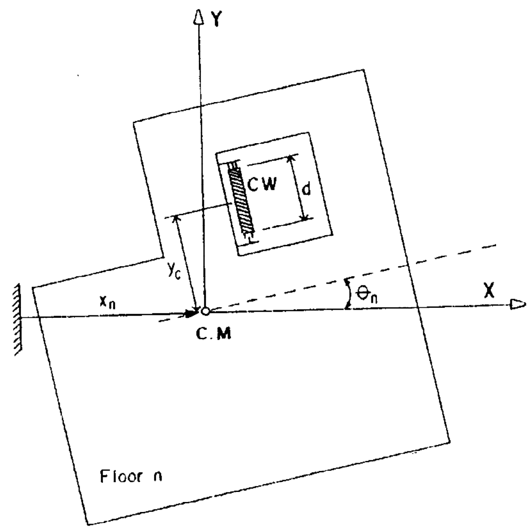

Fig. 2. Plan view of the floor slab and counterweight

The natural frequencies of the uncoupled systems in Eq. (5) are

$$\begin{aligned} \omega_o &= \sqrt{\frac{4k_t}{m}} \\ \Omega_o &= \sqrt{\frac{3}{1+\gamma_1^2}}\,\omega_o \\ \Omega_o' &= \sqrt{\frac{3}{1+\gamma_2^2}}\,\omega_o \end{aligned} \tag{7}$$

Due to the fact that counterweight frames are constructed of standard sizes, it is possible to obtain a relationship between the three natural frequencies. For a typical counterweight, the frequencies are related as follows

$$\begin{aligned} \Omega_o &= 1.694\,\omega_o \\ \Omega_o' &= 1.730\,\omega_o \end{aligned} \tag{8}$$

The stiffness coefficient k_t is calculated as

$$k_t = \frac{k_s k_b (k_s + 2k_w)}{k_s(k_b + k_s + 2k_w) + k_b k_w} \tag{9}$$

in which k_w is the equivalent stiffness of the rubber tires of the rollers, k_s is the stiffness coefficient of the helical steel springs that keep the rollers in contact with the guide rail, and k_b is the equivalent stiffness of the flexible beam representing the guide rail.
The stiffness of the rubber tires can be calculated approximately as (Suarez and Singh, 1996)

$$k_w = 2\,c\,E_r \tag{10}$$

where c is the width of the guide rail (or width of the wheel) and E_r is the modulus of elasticity of the rubber.
Assuming that the CW is symmetrically placed along the span of the guide rail, the coefficient k_b can be calculated as

$$k_b = \frac{12}{(3-4\chi)} \frac{EI}{L^3} \quad ; \quad \chi = \frac{1}{2}(1 - \frac{\ell}{L}) \tag{11}$$

After solving the equations of motion (5), the total displacements of the upper and lower attachment points of the CW are calculated as

$$u_1 = u_c - u_e - u_e' \quad ; \quad u_2 = u_c - u_e + u_e' \tag{12}$$

The deflection of the guide rail at the points of contact with the CW, u_{bi}, and the total deformation of the roller guides, u_{ri}, are calculated in terms of the displacement of the point of attachment of the counterweight as follows

$$u_{bi} = \frac{\varepsilon}{1+\varepsilon} u_i \quad ; \quad u_{ri} = \frac{1}{1+\varepsilon} u_i \quad ; \quad i = 1,2 \tag{13}$$

where ε is the following non-dimensional coefficient

$$\varepsilon = \frac{k_s(k_s + 2k_w)}{k_b(k_s + k_w)} \tag{14}$$

Numerical Example

Using the relationships in Eq. (8) it is possible to calculate via a time history analysis the response spectrum for the relative displacements u_1 and u_2. A set of historic earthquakes (Loma Prieta, El Centro, Parkfield, etc.) was used for this purpose. The 10-story building described in FEMA (1987) was subjected to one of the horizontal components of the Loma Prieta earthquake. The CW was stationed at the top of the building, between the tenth and ninth floor. Figure 3 shows the response spectra for the displacements u_1, u_2 of the points of attachment for a counterweight system with 2% damping subjected to the Loma Prieta accelerogram. Note that the displacements of the upper and lower points are almost indistinguishable in this case.

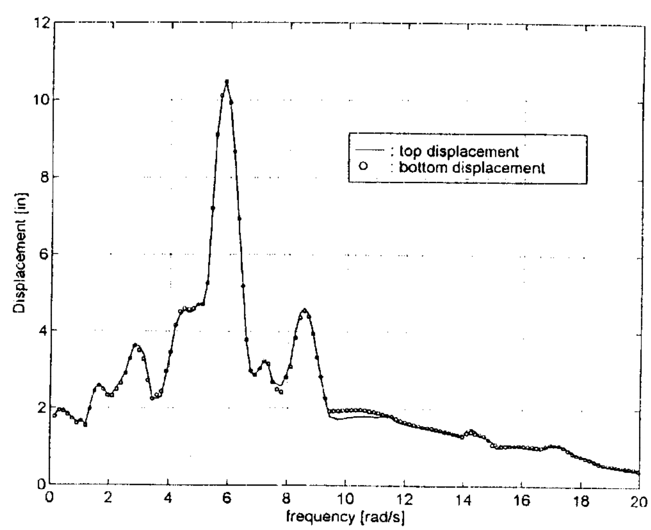

Fig. 3. Displacement floor response spectra for the Loma Prieta earthquake

To calculate the deformation of the guide rail and roller guides using the spectrum, one needs to consider a specific case. For example, consider an 8 lb guide rail for which $I = 1.369\ in^4$ and $c = 0.625\ in$ (ASME, 1990). The dimensions of a typical CW are $d = 28\ in$, $\ell = 138\ in$ and $e = 6\ in$. The weight of the counterweight is 4300 lb. The interstory height, which is assumed to be equal to the length of the guide rail, is $L = 12\ ft$. Taking $E_r = 200\ psi$ and $k_s = 100\ lb/in$, the equivalent stiffness k_t is 171 lb/in.

The natural frequencies of the system are $\omega_o = 7.84\ rad/s, \Omega_o = 13.3\ rad/s$, and $\Omega_o^{'} = 13.6\ rad/s$. The displacements obtained from the spectra are $u_1 \simeq u_2 = 2.5\ in.$ The coefficient ε is equal to $1/735$, which gives a deflection of the guide rail $u_b = 0.0034\ in$ and a deformation of the roller guide $u_r = 2.49\ in.$ Using the above data and considering the rubber tire and the spring that loads the rollers as two series springs, the deformations of the tire and spring are, respectively, 0.71 *in* and 1.78 *in.* Note that because in this case $\ell \approx L$, the stiffness coefficient k_b has a very large value and most of the deformation is sustained by the roller guides.

FLOOR RESPONSE SPECTRUM APPROACH

The analysis of the results obtained with the diverse models showed that in many cases the rotational degrees of freedom (ϕ, θ) or $(u_e, u_e^{'})$ do not play a preponderant role in the response of the counterweight and thus they can be ignored. Therefore, it is possible to study the motion of the CW with a single model dof model. This, in turn, permits to employ a direct floor response spectrum approach to calculate the seismic response of the counterweight (Singh, 1980). Usually floor response spectra are defined in terms of the absolute acceleration of the piece of equipment or non-structural component. Therefore, the formulation needs to be modified since here one is interested in the deformation of the roller guides and guide rails.. Consider a counterweight with natural frequency ω_o and damping ratio ζ_o stationed near the p^{th} floor of an $n-$story torsional building with natural frequencies ω_j , modal damping ratios ξ_j, participation factors $\bar{\gamma}_j$ and vibration modes $\{\phi_j\}$. It can be shown (Suarez and Singh, 1996) that the relative displacement floor spectrum R_d can be calculated as follows:

$$
\begin{aligned}
R_d^2 \; = \; & \sum_{j=1}^{3n} \frac{\bar{\gamma}_j^2 \phi_{\nu,j}^2}{\omega_j^4} [(A_j + r_j^2 B_j) R_j^2 + (C_j - B_j) r_j^4 R_o^2] \\
& + 2 \sum_{j=1}^{3n-1} \sum_{k=j+1}^{3n} \bar{\gamma}_j \bar{\gamma}_k \phi_{\nu,j}^2 \phi_{\nu,k}^2 [(D_{jk} + r_k^2 E_{jk}) \frac{R_k^2}{\omega_k^4} + (F_{jk} + r_k^2 G_{jk}) \frac{R_o^2}{\omega_o^4} \\
& + (H_{jk} - r_j^2 (E_{jk} + G_{jk})) \frac{R_j^2}{\omega_j^4}]
\end{aligned}
\qquad (15)
$$

where $r_j = \omega_j / \omega_k$, and R_o and R_j are, respectively, the acceleration ground response spectrum values for an oscillator with frequencies and damping ratios (ω_o, ζ_o) and (ω_j, ξ_j) . The parameter ν is equal to $3p-2$ or $3p-1$, depending on whether the ground excitation acts parallel to the X or Y axis. The coefficients A_j, B_j and C_j are

$$
\begin{aligned}
A_j &= [-1 + \epsilon_o(\epsilon_j - 4\xi_j^2) r_j^2 + (1 + 4\xi_j^2 \epsilon_j - \epsilon_j^2) r_j^4]\ r_j^4 / \Delta \\
B_j &= [-4\xi_j^2 + \epsilon_o r_j^2 + (4\xi_j^2 - \epsilon_j^2) r_j^4]\ r_j^2 / \Delta \\
C_j &= [4\xi_j^2 \epsilon_o + (1 - 4\xi_j^2 \epsilon_j - \epsilon_o^2) r_j^2 + \epsilon_o \epsilon_j r_j^4 - r_j^6]\ r_j^2 / \Delta
\end{aligned}
\qquad (16)
$$

where

$$
\begin{aligned}
\Delta &= -1 + \epsilon_o \epsilon_j r_j^2 + (2 - \epsilon_j^2 - \epsilon_o^2) r_j^4 + \epsilon_o \epsilon_j r_j^6 - r_j^8 \\
\epsilon_o &= 2\ (4\zeta_o^2 - 1) \quad ; \quad \epsilon_j = 2\ (4\xi_j^2 - 1)
\end{aligned}
\qquad (17)
$$

The definition of the coefficients D_{jk}, E_{jk} ,etc., can be found in the report by Suarez and Singh (1996); they are not included here due to space limitations. Moreover, in some cases (for instance when the structure has well separated frequencies) the terms in the double summation in Eq. (15) can be neglected.

Numerical Example

Consider the same 10-story building of the previous example. This time the seismic input is the NEHRP ground response spectrum for earthquake-resistant design of buildings (Building Safety Officials, 1992) for a soil profile type S_1 and effective peak acceleration $A_a = 0.2$. Figure 4 shows the input ground response spectra for 2 % and 5 % damping ratios.

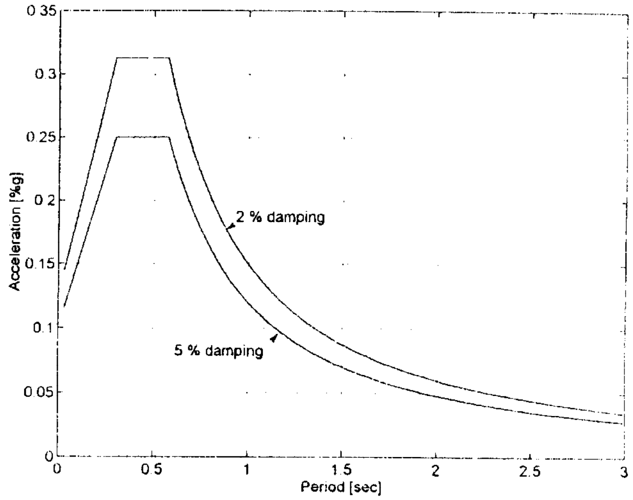

Fig. 4. NEHRP design ground response spectra

Figure 5 shows the relative displacement floor response spectra calculated with Eq. (15) for floors 1, 5 and 10. For the CW of the previous example stationed near the top floor, the deformation of the rail guide and roller guide is 2.8 in.

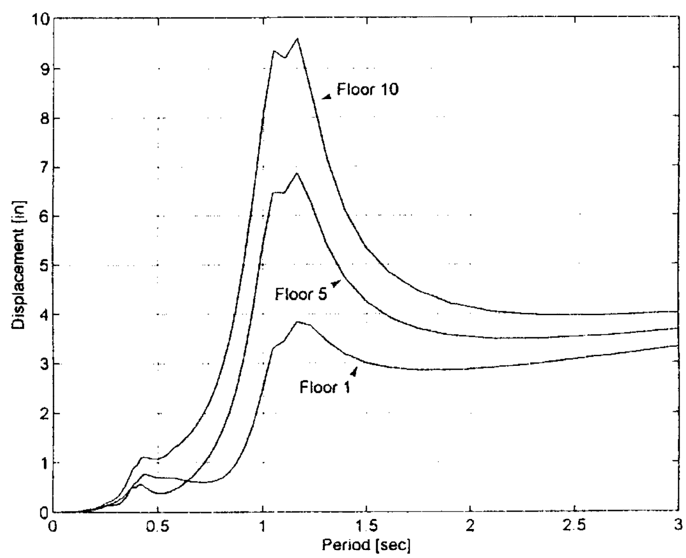

Fig. 5. Displacement floor response spectra for the NEHRP design spectrum

CONCLUSIONS

A procedure to calculate the out-of-plane response of elevator counterweights during earthquakes was presented. The response of a typical CW in a torsional building subjected to the Loma Prieta earthquake was calculated using floor response spectra obtained via time history analysis. The displacements obtained from the spectra can be used to calculate the deflection of the guide rail and the deformation of the rubber tires and springs of the roller guide. The CW response was also computed using a displacement floor response spectrum obtained with a direct method. This method avoids lengthy time history analyses and it can use as seismic input the same ground motion spectra provided by the codes for the design of the building.

The results presented in this paper are part of a comprehensive ongoing study on the effects of earthquakes on elevators. The final goal is to obtain simple yet rational design methodology to help the industry to enhance the safety of elevator passengers and to mitigate the damage to elevators during strong earthquakes.

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