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PREDICTIVE CAPABILITIES OF THE ALPHA METHOD: SHAKING TABLE TESTS AND FIELD DATA VERIFICATION

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SUMMARY

This paper verifies the accuracy and effectiveness of the “alpha” method for maximum rotational response prediction as applied to a wide range of eccentric systems and earthquake excitations.
The verification is carried out either through extensive numerical investigations, through shaking table tests and through the analysis of actual responses of two Californian base isolated structures subjected to some of the most recent earthquakes occurred in their regions. These studies showed the applicability of the proposed “alpha” method which is found to be sufficiently accurate (for engineering purposes) and robust over a wide range of eccentric system parameter values and dynamic excitations. These successful verification results also confirm that the dimensionless structural parameter “alpha” used in the proposed simplified method can alone be used to quantify the pre-disposition of a given eccentric system in developing a rotational response under earthquake excitations.

INTRODUCTION

Using the “alpha” method [1,2], the maximum rotational response, $|u_\theta|_{\max}$, of a given eccentric system subjected to earthquake excitation can be predicted through the following simple formula [1,2,3]:

$$|u_\theta|_{\max} \cong \alpha \cdot \frac{|u_y|_{\max-ne}}{\rho} \tag{1}$$

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where $\left|u_y\right|_{\max-ne}$ is the maximum longitudinal (in the *y*-direction) displacement developed by an SDOF oscillator of same mass and same uncoupled lateral stiffness as the eccentric system, ρ is the mass radius of gyration of the superstructure with respect to the center of mass, and α is the dimensionless purely structural parameter available in exact closed-form for undamped systems and approximate analytical forms for damped systems [1,2].
To verify the validity of the "alpha" method, its degree of accuracy and robustness, estimation of the maximum system rotation obtained using Eq.1 is compared with the "exact" maximum rotational response as obtained through numerical simulation, shaking table tests and field data analysis. The numerical verification is performed over a wide range of eccentric system parameter values. The system parameters consist of the relative eccentricity $e_x = \frac{E_x}{D_e}$ along the transversal (*x*-) direction (E_x being the system transversal eccentricity and D_e the equivalent diagonal, i.e. a reference measure of the system dimensions equal to $\rho\sqrt{12}$), the structural parameter $\gamma = \frac{\omega_\theta}{\omega_L}$ (ω_θ being the natural circular frequency of rotational vibration of a fictitious non-eccentric structure having the same rotational stiffness and mass moment of inertia with respect to the *z*-axis as the eccentric system considered here and ω_L being the uncoupled lateral natural circular frequency of vibration) and the modal viscous damping ratio ξ [1,2]. The experimental verification is based on a large set of shaking table tests performed on a small-scale one-storey building model carefully designed to accommodate a wide range of adjustable eccentricity *e* while maintaining a fixed value of structural parameter γ and ξ [4]. The field data analyses take into account both the actual and accidental system eccentricities.

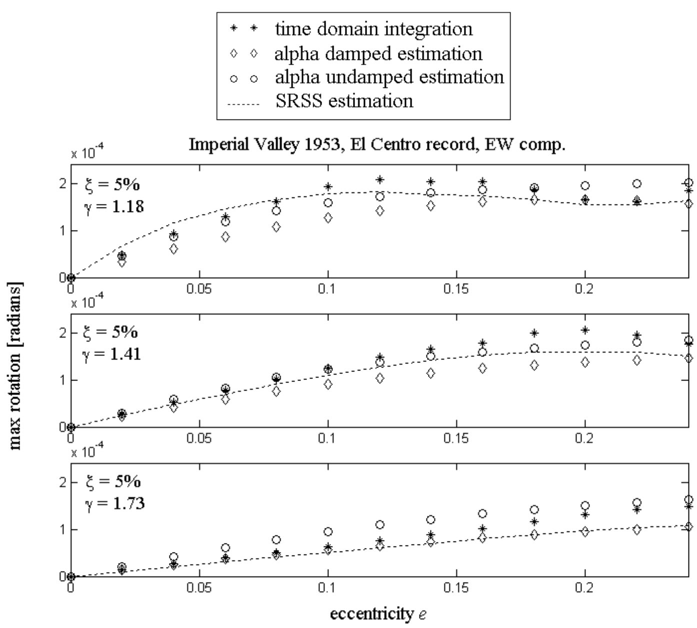
NUMERICAL VERIFICATION OF THE "ALPHA" METHOD

Numerical verification of the "alpha" method is performed through direct numerical integration of the equations of motion of linear elastic one-storey *3*-DOF eccentric systems [1,2], to obtain the "exact" maximum rotational response, $\left|u_\theta\right|_{\max}$. Estimation of $\left|u_\theta\right|_{\max}$ through Eq.1 requires numerical evaluation of the dimensionless rotational parameter α and of $\left|u_y\right|_{\max-ne}$. In each case described below, parameter α is evaluated using either the closed-form expression for undamped systems or the empirical analytical expressions for damped systems [1,2]. For each earthquake record considered in this study, the response parameter $\left|u_y\right|_{\max-ne}$ is obtained through numerical integration of the equation of motion of an SDOF oscillator of the same mass and uncoupled lateral stiffness as the eccentric system.
The numerical verification study encompasses the following six earthquake ground motion records: Kern county 1952 (Taft record, NS component), Imperial Valley 1953 (El Centro record, EW component), Friuli 1976 (Tolmezzo record, NS component), Landers 1992 (Desert Hot Springs record, NS component), Northridge 1994 (Rancho Palos Verdes record, NS component), and Kobe 1995 (Oka record, NS component). The eccentric systems here considered are characterized by the following wide ranges of structural parameter values: $0 \le e \le 24\%$, $5\% \le \xi \le 15\%$, $1.18 \le \gamma \le 1.73$. These ranges of parameter values cover the majority of cases of seismic isolated structures [5,6]. Figures 1 and 2 compare the "exact" maximum rotational response (in radians) obtained through a *3*-DOF numerical response history analysis with the corresponding estimations obtained using (a) the "alpha" method (using α both in the undamped and damped case [1,2], and (b) the SRSS method of modal combination [7]). These figures provide comparative results in terms of maximum rotational response versus relative eccentricity $e = e_x$ for two of the six earthquake records defined above. These numerical results (together with other results not

presented here [2]) show that (*i*) the "alpha" method is sufficiently accurate for engineering purposes and (*ii*) the "alpha" method provides an estimation of the maximum rotational response that is comparable or superior in accuracy to the estimation obtained via the SRSS method of mode combination. For large values of damping ($\xi = 15\%$), the "alpha" method is consistently more accurate than the SRSS method.

It is important to note however that the "alpha" method, which is found overall to be at least as accurate as the SRSS method, presents several advantages over the latter: (a) it does not require the solution of an eigenvalue problem to obtain the natural frequencies and modes of vibration; (b) it reduces the three-dimensional problem to that of an SDOF system and a simple calculation/estimation of the structural parameter $\alpha = \alpha(\gamma, e, \xi)$; for fairly regular structures, γ can be evaluated directly from the basic layout of the eccentric system through simple exact or approximate formulas, and the relative eccentricity e to be used in a simple code-like formula to estimate the maximum rotational response; (c) for a given linear elastic *3*-DOF eccentric system, it provides, at minimum computational cost, immediate insight into the heart of the lateral torsional coupling problem and the resulting effects on the maximum deformation response of the system due to earthquake excitation (e.g., quick comparison of alternative design solutions, because, since it is bounded between 0 for the non eccentric case and 1, the rotational parameter α can be readily used as a formal index for the tendency of a given structure to develop a rotational response under dynamic excitations, and direct estimation of additional deformation in corner isolators due to rotational response effects); and (d) it is perfectly suited for the incorporation of accidental eccentricity in seismic design (generally prescribed to be 5% of the side of the structure, that in the case of a square-plan structure corresponds to $e = 3.5\%$).

Note that the dependence of the maximum rotational response on structural parameter γ has already been recognized indirectly and qualitatively (on a case-by-case basis) by other researchers [7,8]: the "alpha method" formalizes into a simple, physically-based formula the behavioural trends identified in previous research works.



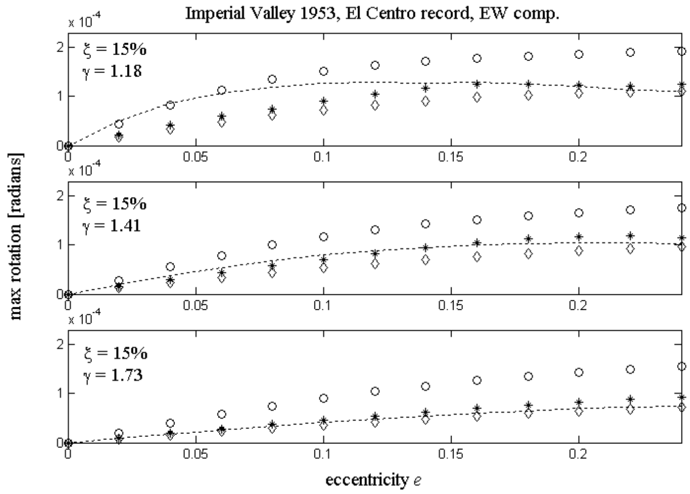


Figure 1: Maximum rotation response vs. eccentricity e for the Imperial Valley 1953 excitation.

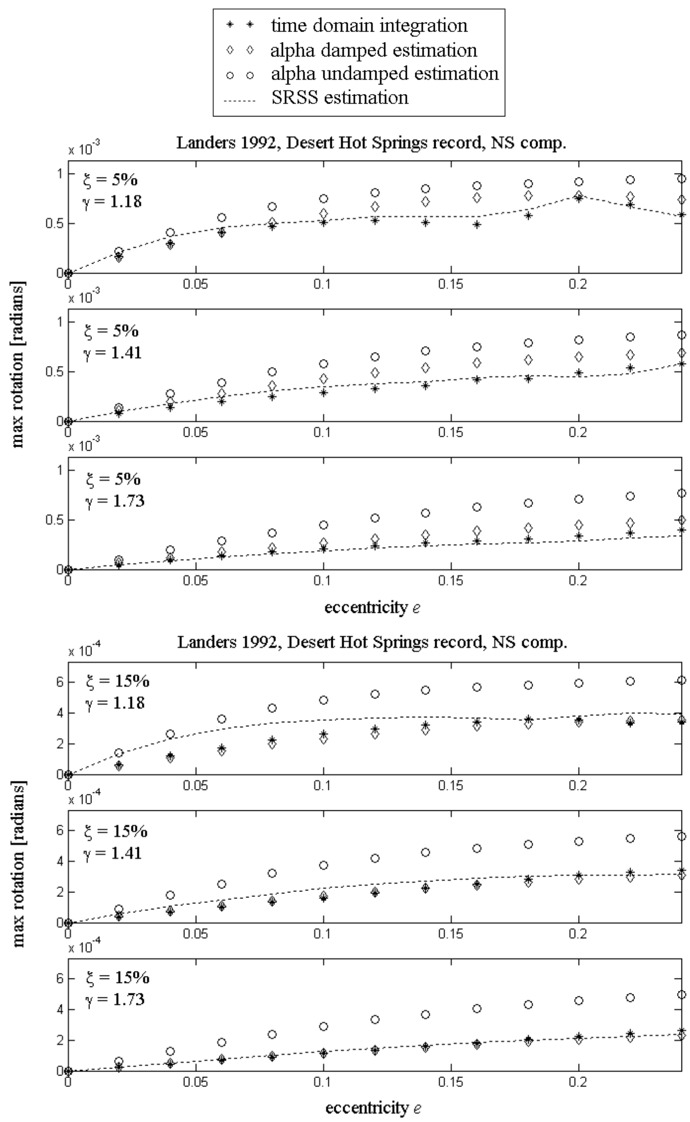


Figure 2: Maximum rotation response vs. eccentricity e for the Landers 1992 excitation.

EXPERIMENTAL VERIFICATION OF THE "ALPHA" METHOD (CIVIL ENGINEERING LABORATORY FACILITY OF RICE UNIVERSITY)

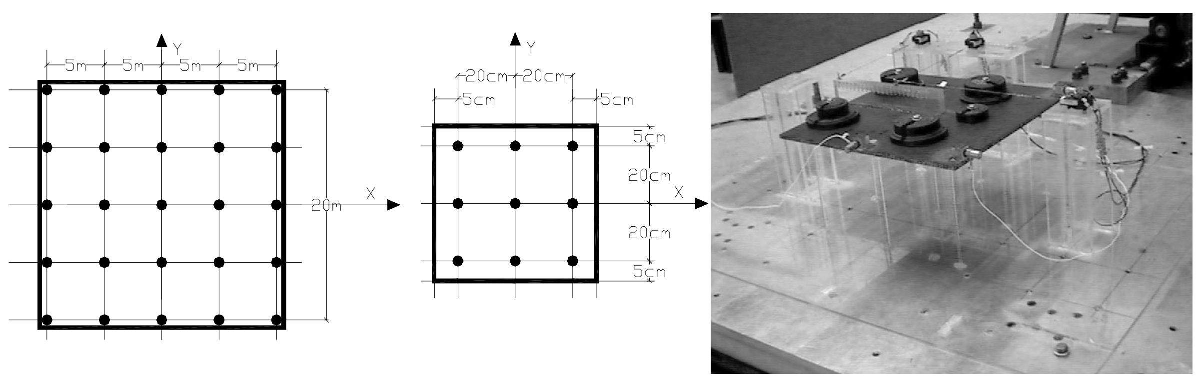
This experimental study consisted of a suite of shaking table tests performed on a versatile small-scale one-storey *3*-DOF building model able to simulate a wide range of eccentric conditions [4].

Targeted prototype structure

In order to design and fabricate a meaningful small-scale model representative of a real seismic isolated building, a prototype structure was selected. A target prototype structure consisting of a four-storey building 20 meters by 20 meters in plan and resting over 25 base isolators located at the nodes of a square 5 m by 5 m grid as shown in Figure 3a is considered in this study, as it represents a fairly common seismic isolated building structure [5,6]. Considering a distributed mass of 1000 kg/m^2 for each floor, the dynamic characteristics of the target prototype structure are summarized in Table-1.

Table-1: Prototype characteristics

Total mass of the superstructure	m = $2 \cdot 10^6$ kg
Polar mass moment of inertia of the superstructure with respect to the center of mass	$I_p = 1.\overline{3} \cdot 10^8 \, kg \cdot m^2$
Mass radius of gyration of the superstructure with respect to the center of mass	ρ = 8.16 m
Uncoupled lateral natural period of vibration	$T_L = 2$ sec
Lateral stiffness in any direction of the 25 isolators combined	$k = 2 \cdot 10^7$ N/m
Rotational stiffness of the total base isolation system about the center of mass	$k_{\theta\theta} = 2 \cdot 10^9$ Nm/rad
Ratio of lateral to rotational natural period for zero eccentricity: γ parameter	$\gamma = \dfrac{\omega_\theta}{\omega_L} = \sqrt{\dfrac{k_{\theta\theta}}{\rho^2 \cdot k}} = 1.225$
Rotational natural period of vibration for zero eccentricity	$T_\theta = 1.632$ sec



(a) (b) (c)

Figure 3: (a) Plan view of the prototype building. (b) Plan view of the model. (c) The reduced-scale model as built.

Model structure

To accommodate the characteristics of the Rice University shaking table on which these tests were performed [4], for ease of response data acquisition, and given the characteristic of the theory to be verified, it has been constructed a linear elastic one-storey building model with null longitudinal eccentricity ($e_y = 0$), with adjustable distance between the center of mass and the center of stiffness (i.e., system eccentricity) along the transversal (x-) direction and with other dynamic characteristics kept constant. Thus the model had to satisfy the following requirements:

- time scale factor between prototype and model $\lambda_T = T_p / T_m = 5$, where T_p and T_m are the natural periods of vibration of the prototype and the model, respectively;
- length scale factor between prototype and model $\lambda_L = L_p / L_m \cong 40$, where L_p and L_m are the side lengths of the prototype and the model, respectively;
- maintain a linear elastic behaviour throughout the shaking table tests;
- have a precisely located and fixed center of stiffness;
- have a precisely located and movable center of mass;
- have a precisely defined mass radius of gyration for the case of zero eccentricity;
- have a constant value of its uncoupled lateral natural period of vibration throughout the shaking table tests;
- have a constant value of the dimensionless structural parameter γ.

In order to satisfy the above requirements, a model was built consisting of a carbon fiber sandwich plate supported by nine (three rows of three) solid plexiglas column rods fixed to both the carbon fiber top plate and a plexiglas base plate from the above scaling factors, the model must have an uncoupled lateral natural period of vibration of $T_L = 0.40\,\text{sec}$ sec, a mass radius of gyration of about $\rho = 20.4$ cm, and a parameter $\gamma = 1.225$. The carbon fiber sandwich top plate is squared 50 cm by 50 cm in size and weighs 0.550 kg. The nine plexiglas column rods are 6.35 mm in diameter, 225 mm in length and are located at the nodes of a 20 cm by 20 cm square grid, as it is shown in Figures 3b and 3c. The plexiglas columns were sized, assuming a Young's modulus for the plexiglas of 294300 N/cm^2, to achieve a lateral stiffness of the model of 2000 N/m, which leads to a model uncoupled lateral natural period of 0.40 sec for a total model mass of 8.00 kg (including the added masses used to create the mass eccentricity with respect to the center of stiffness). Four weights of 1.850 kg each (including the clamping bolts) are fixed to the top carbon fiber sandwich plate in various configurations. The locations of the added weights were carefully computed in order to obtain precisely transversal relative eccentricities of $e = e_x =$ 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, and 20 percent, while maintaining a constant value of parameter $\gamma = 1.225$. The static and dynamic characteristics of the model, as experimentally measured, are summarized in Table-2. The actual uncoupled lateral natural frequency of the model is 2.2 Hz, which is 12% lower than the corresponding target design value due to the smaller stiffness and larger mass of the model as compared to the target design values. The time scaling factor $\lambda_T = T_p / T_m = 5$ selected for this experimental dynamic study gives a prototype uncoupled lateral natural frequency of 0.44 Hz. This prototype dynamic characteristic, although differing from the targeted prototype value of 0.5 Hz, is nonetheless still representative of common real seismic isolated buildings, which validates the model constructed for the purpose of the present investigations.

Table-2: Static and Dynamic Characteristics of the Model

Lateral stiffness	$k = 1800$ N/m
Resonant frequency (corresponding to the peak in the magnitude transfer function and a phase lag of π/2 in the phase transfer function)	$f_L = \dfrac{\omega_L}{(2\pi)} = 2.2\,\text{Hz}$
Damping ratio of the system in the uncoupled longitudinal mode of vibration obtained through the half power bandwidth method	$\xi = 6\%$
Uncoupled rotational natural frequency	$f_\theta = \dfrac{\omega_\theta}{(2\pi)} = 2.75\ \text{Hz}$

Parameter γ (very close to the target design value)	$\gamma = \frac{\omega_\theta}{\omega_L} = \frac{f_\theta}{f_L} = \frac{2.75}{2.20} = 1.25$

Testing procedure

The small-scale model of an eccentric system described above was tested on the Rice University uni-axial shaking table controlled to reproduce the earthquake records given in Table-3. Note that different length scalings were used for the selected earthquake records, so as to obtain scaled ground (table) acceleration records with a peak ground (table) acceleration in the range between 0.1g and 0.15g. This range was selected in order to induce a relative displacement response of the model that is large enough to be measured accurately by the displacement transducers, but small enough not to threaten the structural integrity of the model. The model was tested for the 8 earthquake records presented in Table-3 and for 11 different added weight configurations in order to increase progressively the transversal (*x*-) eccentricity between the center of mass and the center of stiffness from 0% (non eccentric structure) to 20% of the equivalent diagonal D_e of the model ($0 \le e \le 0.20$, $\Delta e = 0.02$).

Table-3: Time and Amplitude Scaling Factors for Earthquake Records

EARTHQUAKE RECORD	*Time Scale* λ_T	*Length Scale* λ_L
Parkfield 1966	5	41.7
El Centro 1940	5	20.8
Montenegro Bar 1979	5	20.8
Montenegro Petrovac 1979	5	16.7
Friuli Breginj 1976	5	8.3
Synthetic earthquake from Eurocode	5	8.3
Tolmezzo 1977	5	8.3
Brienza 1977	5	8.3

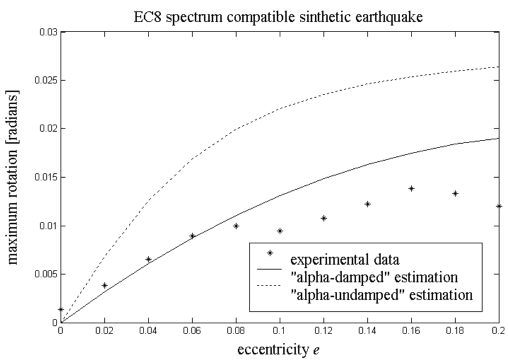
Test results

As the main objective of the experimental tests is to validate the accuracy and robustness of the proposed simplified method, the maximum - experimentally determined - rotational response, $|u_y|_{\max-\exp}$, was compared with the corresponding estimation obtained using Eq.1 where $|u_y|_{\max-ne}$ was also experimentally determined by testing the structural model under condition of null eccentricity. Note that in Eq.1, $|u_y|_{\max-ne}$ together with the mass radius of gyration ρ can be introduced either at the prototype or at the model level; the rotational response $|u_\theta|_{\max}$ in radians is the same at the prototype and model levels. The rotational α parameter is only a function of the relative eccentricity *e*, the structural parameter γ (kept constant at for all experiments) and the modal damping ratio ξ [1,2]; moreover equal modal damping ratios were assumed for all three modes of vibration, being this assumption reasonable in seismic isolated structures. The modal damping ratio is here taken equal to 6%, which is the modal damping ratio identified experimentally for the uncoupled longitudinal mode of vibration under null eccentricity condition. Two values of α were used in Eq.1 to estimate the maximum rotation: (a) the so called "alpha undamped" (α_u), as obtained from closed-form expression [1,2] and (b) the so called "alpha damped" (α_d), as obtained through least square fitting of the results of numerical simulations of damped free vibrations [1,2]:

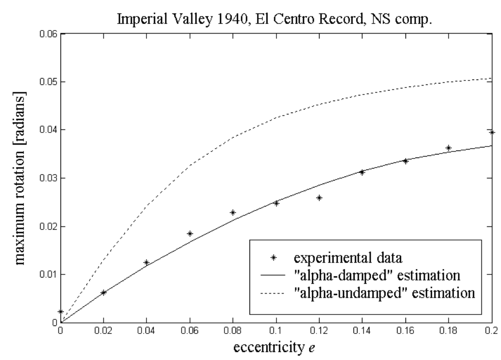
$$\alpha_d = -0.70 \cdot e + 10.25 \cdot \frac{e}{\gamma^2} - 30.70 \cdot \frac{e^2}{\gamma^4} \qquad \xi = 6\% \qquad (2)$$

Figures 4a through 4f compare measured maximum rotational responses, plotted as a function of the relative eccentricity *e*, with their corresponding estimations (both α_u and α_d estimations) obtained using the proposed "alpha" method. As in the numerical verification, these experimental results confirm that the maximum rotational response estimated using the proposed "alpha" method is generally sufficiently accurate for engineering purposes. For structural parameter $\gamma = 1.25$ considered here, the "alpha" method (using α_d, for the damped case) is particular accurate at levels of relative eccentricity *e* ranging from 0 to 8%. Moreover, it has to be noted the small but non zero maximum rotational response obtained experimentally for zero nominal eccentricity due to a small accidental eccentricity of the model. It is also worth pointing out that the "alpha" method (using α_u) provides a conservative upper bound envelope for the maximum rotational response.

Figure 5 shows the measured maximum longitudinal displacement response, $\left|u_y\right|_{\max}$, at the center of mass of the model as a function of the transversal relative eccentricity *e* for each of the 8 earthquake records considered. It is observed that, for a given base excitation, $\left|u_y\right|_{\max}$ does not change significantly with increasing eccentricity *e* and can be reasonably approximated by the maximum longitudinal displacement response for null longitudinal eccentricity, $\left|u_y\right|_{\max-ne}$.



(a)



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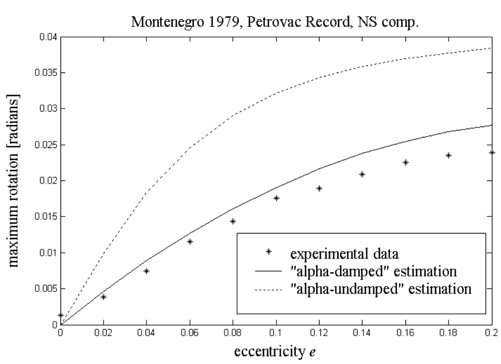

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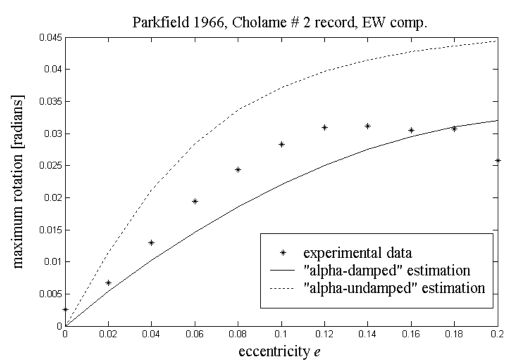

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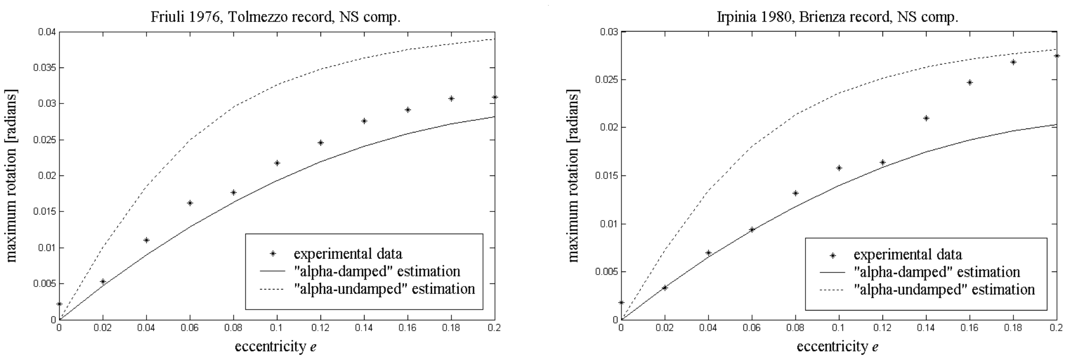


(e) (f)

Figure 4: Experimentally observed maximum rotational response and their corresponding estimations by the "alpha" method ($\gamma = 1.25$, $\xi = 6\%$), for (a) an EC8 Spectrum compatible synthetic earthquake, (b) the Imperial Valley 1940, El Centro earthquake record (N-S component), (c) the Montenegro 1979, Petrovac earthquake record (N-S component), (d) the Parkfield 1966, Cholame #2 earthquake record (E-W component), (e) the Friuli 1976, Tolmezzo earthquake record (NS component) and (f) the Irpinia 1980, Brienza earthquake record (NS component).

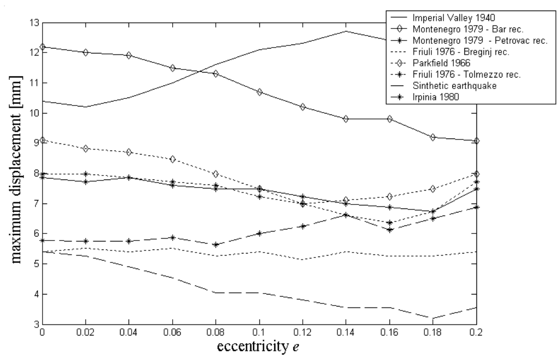


Figure 5: Measured maximum longitudinal displacement at the center of mass of the reduced-scale model for increasing transversal eccentricity e.

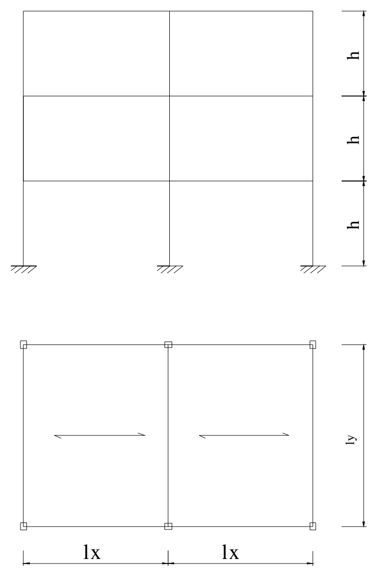
EXPERIMENTAL VERIFICATION OF THE "ALPHA" METHOD (CIVIL ENGINEERING LABORATORY FACILITY OF BRISTOL UNIVERSITY)

This experimental study consisted of a suite of 4 shaking table tests performed on two building models [11]. The first one was a symmetric three-storey 1/5-scale model with respect to its prototype. The second model was asymmetric; the asymmetry was obtained by shifting the mass centre CM of the symmetric model from the stiffness centre CS by a distance e equal to 10% of the plan dimension along the x-direction. In the following, the characteristics of the asymmetric model will be represented.

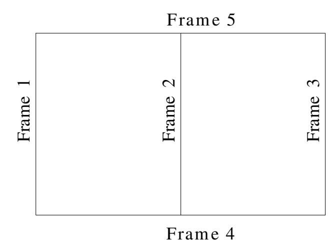
Asymmetric model structure

In order to obtain the fixed value of mass eccentricity, while keeping the total mass and mass radius of gyration about CM constant, the mass distribution corresponding to the presence of a composite steel-concrete floor slab and to the live loads specified by Eurocode 8, was characterized by the parameters reported in Table-4. This mass distribution closely matches the target mass eccentricity and mass radius.

The above parameters were matched in laboratory by using a steel plate (150x200 cm) with thickness of 10 mm and by properly distributing 100 lead blocks, each of them weighing 25 kg, as shown in Figure 6, where CS and CM denote stiffness and mass center, respectively.
For the sake of simplicity in model construction, it was decided that the column sections remain constant along the height. As represented in Figure 7, all beams were square hollow sections 40x40, with t = 30 mm; all column sections are constant along the height (corner columns of rectangular hollow sections 60x40, t = 3 mm, central columns of square hollow sections 50x50, t = 3 mm).



PROTOTYPE	MODEL
lx = 5000 mm	lx = 1000 mm
ly = 7500 mm	ly = 1500 mm
h = 3500 mm	h = 700 mm



Asymmetric Model

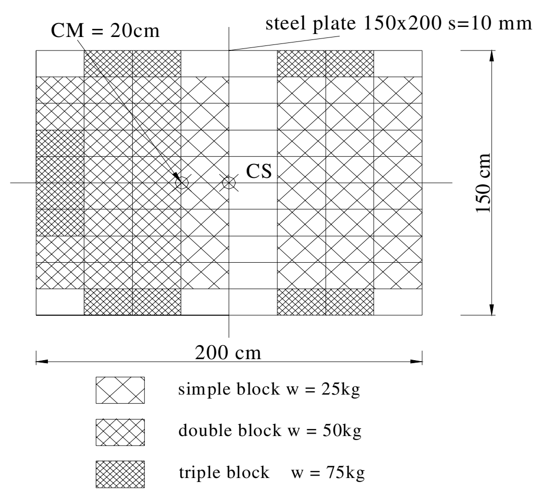


Figure 6: Prototype and model geometrical characteristics and plan view of the model.

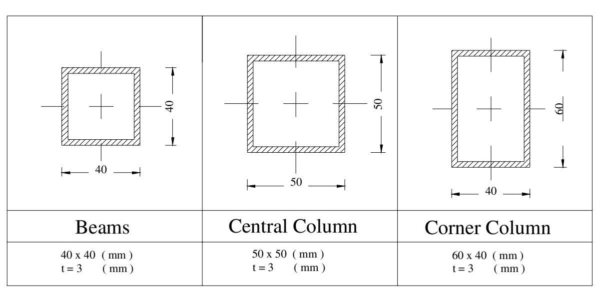


Figure 7: Beams and columns sections

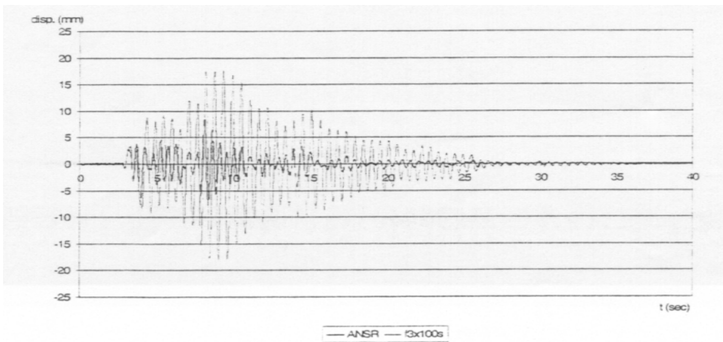
Table-4: Mass distribution

Mass eccentricity	20.56 cm
Total story weight	26.84 kN
Translational mass	0.02736 kNsec^2/cm
Rotational mass	142.9 kNsec^2/cm

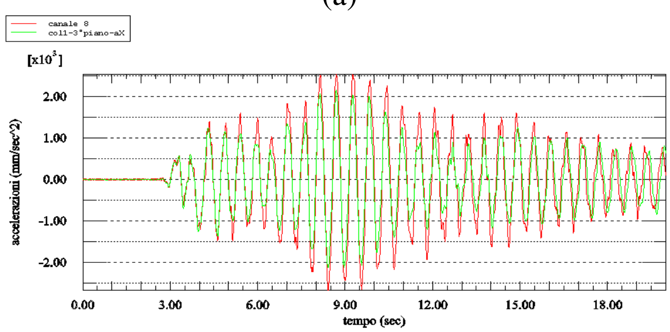
Testing procedure

The 1/5 scale model of the eccentric system described above was tested on the Bristol University shaking table controlled to reproduce the seismic input of the two horizontal components of El Centro Earthquake (1940), named ELCS00E (Peak Ground Acceleration = 0.348 g) and ELCS90W (Peak Ground Acceleration = 0.214g). The simulations were conducted both with the real earthquake time histories and with the ones obtained through ABAQUS schematization of the model, represented in Figure 8.

The model was instrumented with 10 accelerometers, 12 displacement transducers and 25 strain gauges.



(a)



(b)

Figure 8: (a) El Centro time history (b) ABAQUS simulation

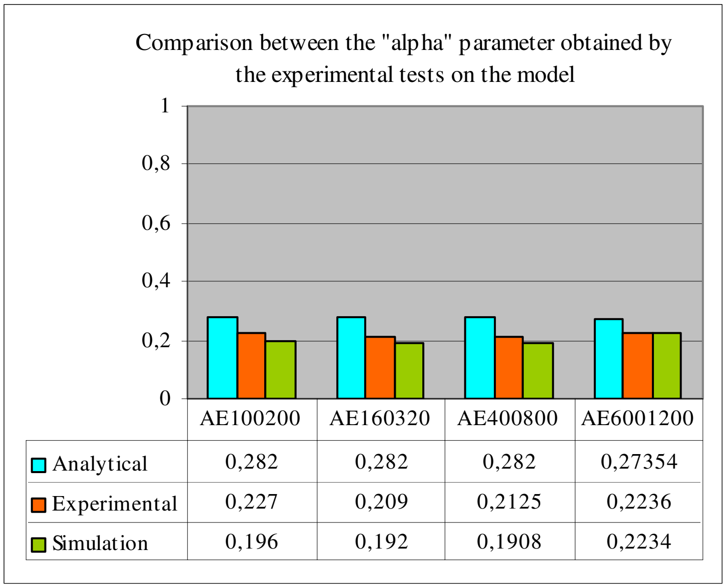
Test results

As the main objective of the experimental tests is to validate the accuracy and robustness of the proposed simplified method, the maximum - experimentally determined - rotational response, $\left|u_y\right|_{\max-\exp}$, was compared with the so called "alpha damped" (α_d), as obtained through least square fitting of the results of numerical simulations of damped free vibrations [1,2]:

$$\alpha_d = -1.74 \cdot e + 15.71 \cdot \frac{e}{\gamma^2} - 51.17 \cdot \frac{e^2}{\gamma^4} \qquad \xi = 2\% \qquad (3)$$

The modal damping ratio was taken equal to 2%, because the damping ratio of the real structure on which the model was constructed, was assumed to be 2.45%

The comparison between the experimental and the analytical results, represented here in Figure 9, confirm that the maximum rotational response estimated using the proposed "alpha" method is higher than the values obtained with the simulations conducted, thus considering this method sufficiently accurate for engineering purposes.



	AE100200	AE160320	AE400800	AE6001200
Analytical	0,282	0,282	0,282	0,27354
Experimental	0,227	0,209	0,2125	0,2236
Simulation	0,196	0,192	0,1908	0,2234

Figure 9: Comparison between "alpha" values

FIELD DATA ANALYSIS

In order to validate the "alpha" method, a number of field data has been analysed. Two base isolated buildings placed in California were taken into consideration: the Foothill Communities Law and Justice Center, Rancho Cucamonga, San Bernardino County and The Los Angeles County Fire Command and Control Facility (hereafter referred to as RCLJCB and LA2FCCFB, respectively), shown in Figures 10a and 10b [9,10].

For these structures, field response data were available for ground motion events occurred in the last twenty years, such as Northridge (1992), Landers (1992), Upland (1990), Whittier (1987) and Redlands (1985), whose main characteristics are summarized in Table-5 [9,10]. These buildings were in fact instrumented with several accelerographs located at different levels: (a) at foundation level below the base isolation system, (b) at basement level above the base isolation system, and (c) at roof plan, see Figure 11.

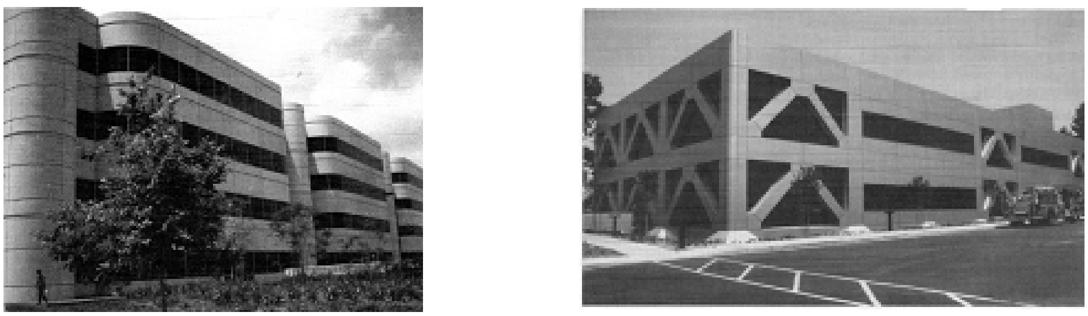
The verification of the "alpha method" was obtained through the comparison between the "alpha" values as computed from the geometrical and mechanical characteristics of the examined real systems, and the $\rho\left(\frac{|u_\theta|_{\max}}{|u_y|_{\max}}\right)_{forced}$ ratios measured from vectorial analysis (conducted considering the real direction of earthquake inputs) of the response time histories given by the sensors located inside the structures. In this analysis, the intrinsic eccentricity of the two structures has been computed from geometrical data (the lateral stiffness of all isolators was considered to be the same). An accidental eccentricity equal to 5% of the side of the building was added to the intrinsic one, in accordance with the indications of the majority of building codes. In the results presented the experimental data are compared to the corresponding "alpha damped” estimation obtained through Eq.1 with α computed using $e=\sqrt{e_y^2+e_x^2}$, $u_{\max}=\sqrt{u_{y\max}^2+u_{x\max}^2}$ and $\xi=12\%$ (see Figure 12 and Figure 13).

Both deformations and rotations of the isolators have been computed through comparison of the response time histories (as developed by J. Stewart [9]) recorded above the isolators and the foundation levels, thus considering the behaviour of the superstructure as a rigid deck.

From this comparison, Fig. 14 shows that the maximum value obtained by the field data records of every examined earthquake is lower than α_d , as given by the following analytical equation [1,2], considering that the isolators together with the ground provide the systems with a reasonable damping of $\xi=12\%$:

$$\alpha_d=-0.23\cdot e+6.59\cdot\frac{e}{\gamma^2}-18.19\cdot\frac{e^2}{\gamma^4} \qquad \xi=12\% \qquad (3)$$

The relative low accuracy of prediction is nonetheless considered acceptable in the light of the numerous approximations and uncertainties present in this study (estimation of accidental eccentricity, not unidirectional seismic input, presence of the here neglected longitudinal eccentricity, etc.).



(a) (b)

Figure 10: Picture of (a) RCLJCB and (b) LA2FCCFB.

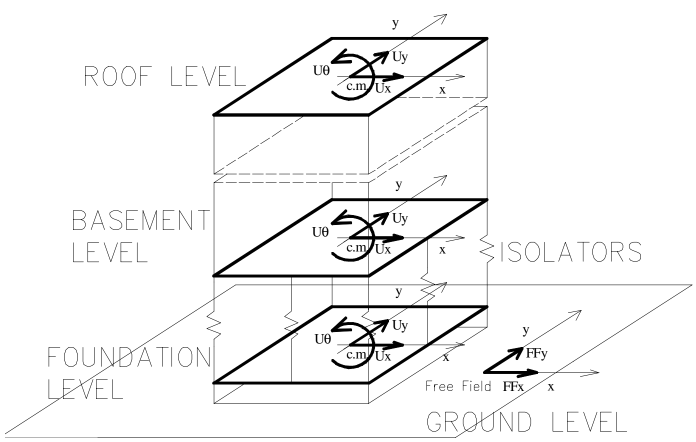


Figure 11: Location of sensors inside the two buildings

Table-5: Earthquakes Characteristics

Peak Ground Accelerations (PGA)						
RCLJCB	Free Field			Foundation Field		
	NS	*EO*	*Vertical*	*NS*	*EO*	*Vertical*
Upland 1990	0.24 g	0.23 g	0.16 g	0.14 g	0.11 g	0.084 g
Landers 1992	0.11 g	0.068 g	0.048 g	0.097 g	0.064 g	0.053 g
Northridge 1992	0.072 g	0.046 g	0.032 g	0.039 g	0.037 g	0.029 g
Redlands 1985	0.040 g	0.032 g	0.027 g	0.028 g	0.035 g	0.015 g
Whittier 1987	0.046 g	0.057 g	0.035 g	0.019 g	0.028 g	0.020 g
LA2FCCFB	Free Field			Foundation Field		
	NS	*EO*	*Vertical*	*NS*	*EO*	*Vertical*
Landers 1992	0.057 g	0.053 g	0.032 g	0.052 g	0.051 g	0.024 g
Northridge 1992	0.22 g	0.32 g	0.13 g	0.17 g	0.22 g	0.11 g

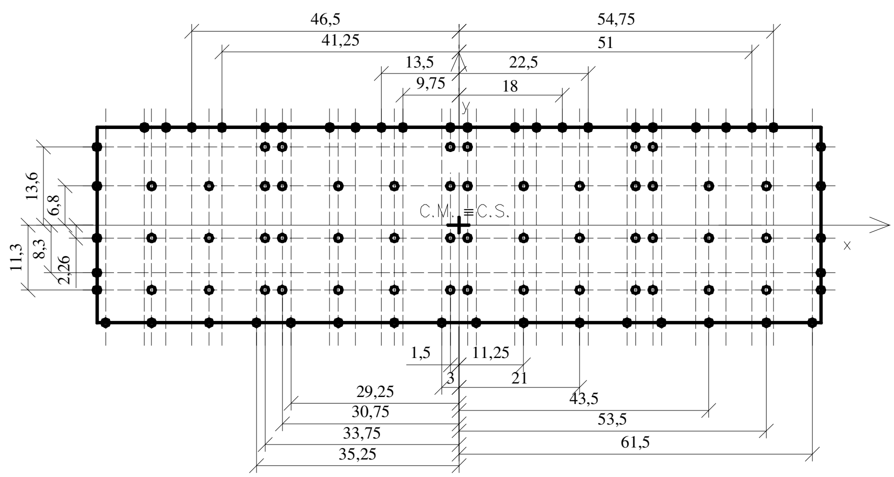


Figure 12: Isolators plan location of RCLJCB, used to calculate the eccentricity *e* and the structural parameter γ ($e_y = 0.024$, $e_x = 0.054$, $e = \sqrt{e_y^2 + e_x^2} = 0.059$, $\gamma = 1.07$)

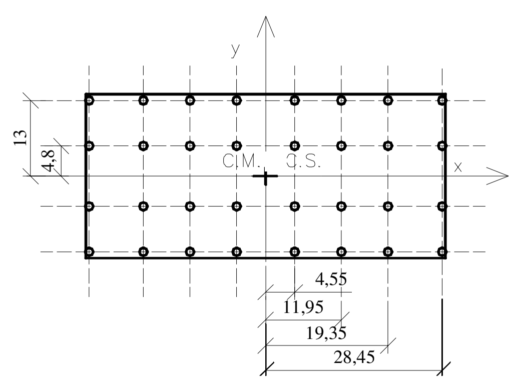
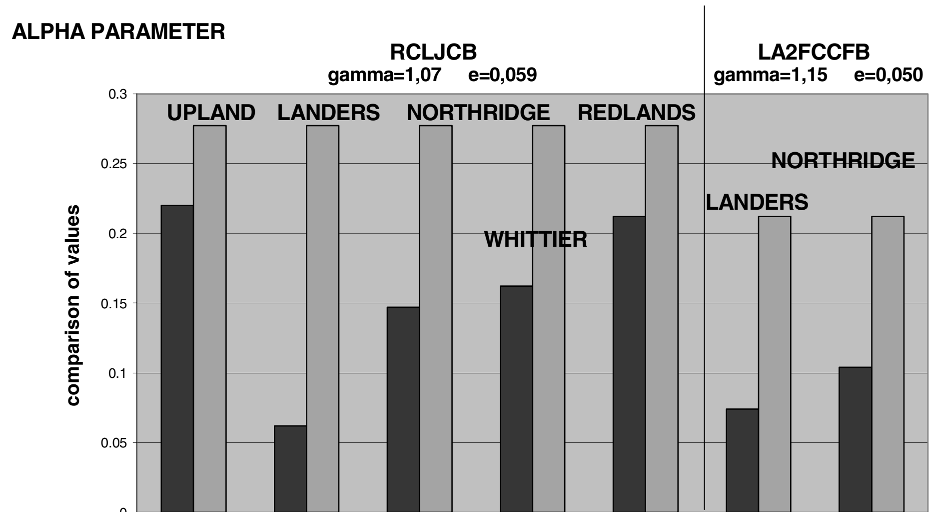


Figure 13: Isolators plan location of LA2FCCFB, used to calculate the eccentricity e and the structural parameter γ ($e_y = 0.021$, $e_x = 0.045$, $e = \sqrt{e_y^2 + e_x^2} = 0.050$, $\gamma = 1.15$)



	1	2	3	4	5	6	7
alpha ass.	0.22	0.062	0.147	0.162	0.212	0.074	0.104
alpha d12%	0.277	0.277	0.277	0.277	0.277	0.212	0.212

Figure 14: Comparison between the maximum values of the α parameter obtained by the field data records and the $\alpha_d\left(\xi = 12\%\right) = \alpha_{d\ 12\%}$.

CONCLUSIONS

This paper presents the results of a numerical and experimental verification study for the simplified approach (“alpha” method) to the seismic response analysis of linear elastic one-storey structural systems with non-coincident center of mass and center of stiffness; seismic isolated building structures representing an important class of such eccentric systems.

The numerical verification was carried out using 6 historical earthquake records as base dynamic inputs and over a wide range of structural parameter values encompassing a wide spectrum of design situations for seismic isolated structures. The numerical verification results obtained in this study show that the “alpha” method (a) is sufficiently accurate for engineering purposes to estimate the maximum rotational response developed by an eccentric system under seismic excitation, (b) is robust to a wide spectrum of system parameter values, and (c) is at least as accurate as the widely used SRSS modal combination method for maximum rotational response prediction.

The experimental verification was performed through shaking table tests conducted at Rice and Bristol University of a small-scale building model. The experimental tests (a) confirm the results of the numerical verification study in terms of accuracy and robustness of the “alpha” method, and (b) validate the simplifying assumptions and behavioural trends used and identified, respectively, in developing the simplified “alpha” method.

The accuracy and robustness of the “alpha” method for maximum rotational response prediction, were also verified through the analysis of the actual responses of two Californian real base isolated structures subjected to earthquakes occurred over the last twenty years.

In summary, the above numerical and experimental verification studies and field data analysis show that the simplified “alpha” method satisfies the engineering requirements of accuracy and robustness. The proposed “alpha” method has the following significant advantages over alternative methods, namely (a) it reduces the *3*-DOF problem to that of an equivalent SDOF system with the additional calculation of the rotational parameter α which depends on structural parameters γ, e (relative eccentricity), and ξ (modal damping ratio) [1,2]; and (b) it provides, at minimum computational cost, immediate insight into the heart of the lateral-torsional coupling problem through the identification of the controlling structural parameters and the sensitivities of the maximum rotational response of the structure to these parameters. Therefore, the “alpha” method can be readily and effectively used for practical design purposes. In particular, it is perfectly suited for the incorporation of accidental eccentricity in seismic analysis and design.

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