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Considering earthquake direction on seismic analysis

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ABSTRACT: A method to include earthquake directional effects on the seismic analysis of building is presented. A comparative study to examine the accuracy of the method is performed. Three linear buildings with different structural characteristics were analysed considering actual earthquake acceleration records and acceleration response spectra, and acceleration design spectra. Various techniques for combining modal response were considered. It was concluded that the proposed method estimates very well the structural response when the square root of sum of squares (SRSS) technique is applied. The main advantage of the proposed method is that it avoids the use of arbitrary two orthogonal directions on seismic analysis of buildings. The method also evade the need for the standard combination procedure of the earthquake orthogonal effects to obtain the design stresses.

1 INTRODUCTION

As actual earthquakes may have different direction of incidence, and because the three-dimensional nature of buildings causes translational coupling, many building codes indicate that the design stresses must be computed combining the effects of a ground acceleration taking place in any two orthogonal directions of incidence. Furthermore an independent seismic analysis is to be made for each direction (IAEE 1988). The criteria recommended by the building codes for combining those two effects, in some cases may not be good enough (Guendelman et al. 1991).

In this work a method that takes into account the direction of incidence of the ground acceleration on seismic design of building, is presented. The method consists in determining the maximum response of each mode of vibration of the structure, using as parameter the direction of incidence of the ground acceleration. Then those maximum values are combined using any modal combination criterium.

A comparison was made between the results obtained using the proposed method and those obtained using a time step integration method and also using spectral seismic analysis. Three different buildings were utilized in this exercise. The buildings were subjected to actual earthquake ground acceleration record and acceleration response spectra, and also the buildings were subjected to acceleration design spectra recommended by various building codes.

2 STRUCTURAL MODEL

A linear n-storey building model with 3n-degrees of freedom and with viscous damping is analyzed. The model comprises n-rigid decks supported on massless axially inextensible columns. The columns have translational stiffness in two-orthogonal directions. Their torsional stiffnesses have been neglected. The 3n-degrees of freedom of the structural system are: the horizontal displacements (u_j, v_j; $j = 1,2,\ldots,n$) of the centers of mass of the n-rigid decks relative to ground, and their rotations about a vertical axis (θ_j; $j = 1,2,\ldots,n$) (see Fig. 1).

The parameters describing the model are referred to the centers of mass of the rigid decks, which lie on the vertical axis Z. The origin of the coordinate system is located on the intersection of the vertical axis Z and the basal plane. This model is characterized by the stiffness matrix [K], that refers to the 3n-degrees of freedom of the system, and by the matrix of mass [M], which is a diagonal matrix, as it is assumed that the masses of the structure are lumped on the homogeneous rigid decks.

3 SEISMIC RESPONSE OF THE STRUCTURE

The equation of motion for the structural model subjected to a ground acceleration is:

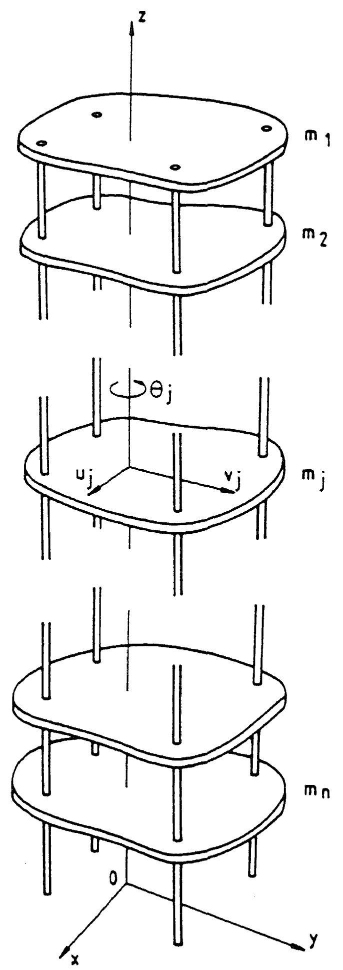

Figure 1. Structural model.

$$[M]\{\ddot{U}\}+[C]\{\dot{U}\}+[K]\{U\}=-[M]\;[G]\{\ddot{U}_g\} \qquad (1)$$

where $\{\ddot{U}_g\}$ is a vector containing the three ground acceleration components (i.e., the two orthogonal translation components and a rotational component about Z), [C] is the matrix of viscous damping, and [G] is a geometrical transformation matrix of the ground displacements. It is assumed that the damping matrix satisfies Rayleigh hypothesis.

By separation of variables:

$$\{U\}=[\phi]\{Y\} \qquad (2)$$

where $[\phi]$ is the modal matrix and vector {Y} is the temporary solution of the differential equation system (Eq.1). Thus the following uncoupled system of equations in the time domain may be obtained:

$$\{\ddot{Y}\}+[\Lambda]\{\dot{Y}\}+[\omega^2]=[B]\{\ddot{U}_g\} \qquad (3)$$

where $[\omega^2]$ is a diagonal matrix that contains the 2nd powers of the natural frequencies of the system, $[\Lambda]$ is a diagonal matrix that contains the viscous damping parameters of each natural mode of vibration of the structure and matrix [B] may be expressed in the following form:

$$[B]=-([\phi]^T[M]\;[\phi])^{-1}[\phi]^T[M]\;[G] \qquad (4)$$

The ith-equation of the system (Eq. 3) may be described in the following form,

$$\ddot{Y}_i+2\lambda_i\omega_i\dot{Y}_i+\omega^2{}_iY_i=(B_{i1}\ddot{U}_g+B_{i2}\ddot{V}_g+B_{i3}\ddot{\theta}_g) \qquad (5)$$

and its solution is

$$Y_i=(B_{i1}\ddot{U}_g+B_{i2}\ddot{V}_g+B_{i3}\ddot{\theta}_g)*h_i \qquad (6)$$

in which * indicates a linear convolution operation and h_i is the response to the unit pulse of the ith-mode of vibration of the system.

3.1 Critical Modal Direction

Critical modal direction is defined as the direction of incidence of an unidirectional ground acceleration which produces the largest dynamic response for that mode.

Neglecting the torsional component of the ground acceleration and assuming the angle of incidence of the unidirectional ground acceleration ($\ddot{u}_g$) as the angle α, measured anticlockwise from the X-axis of the coordinate system (see Fig.2), the dynamic response in time for each mode (Eq.6), is given by:

$$Y_i=(B_{i1}\cos\alpha+B_{i2}\sin\alpha)\,\ddot{u}_g*h_i \qquad (7)$$

By partial derivation of Eq.7 with respect to α and equating to zero, the critical direction of the ith-mode of vibration is obtained (González 1987). Its tangent is given by:

$$\tan\alpha_{ci}=\frac{B_{i2}}{B_{i1}}=\frac{\sum_{k=1}^{n}m_k\nu_{ki}}{\sum_{k=1}^{n}m_k\mu_{ki}} \qquad (8)$$

where μ_{ki} and ν_{ki} represent the X and Y

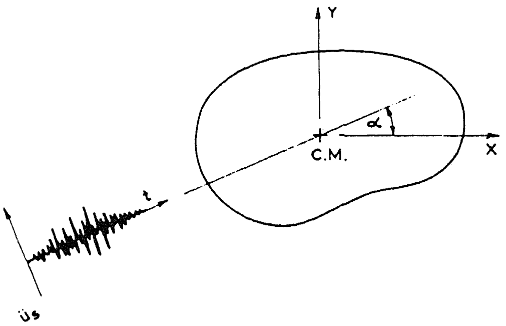

Figure 2. Direction of incidence of an unidirectional ground acceleration.

components of the modal vector i, related to the kth-floor.

4 THE MAXIMUM DYNAMIC MODAL RESPONSE OF THE STRUCTURE

To obtain the maximum response from each mode of vibration the direction of incidence of the earthquake is taken as parameter. Equation 8 defines the angle of incidence of the ground acceleration maximizing the modal response (α_{ci}). Thus the modal response in the time domain can be expressed as a function of α_{ci} in the following form:

$$Y_i=\sqrt{B_{i1}^2+B_{i2}^2}\cos(\alpha-\alpha_{ci})\chi_i \tag{9}$$

where $\chi_i = \ddot{u}_s * h_i$

On the other hand, the maximum value of χ_i corresponds to the relative displacement response spectrum related to the ith-mode (S_{di}). Therefore the maximum modal response of the structure for ith-mode may be obtained by replacing α and χ_i by α_{ci} and S_{di} respectively in Eq.9. This is a maximum value in time and space and it can be written as,

$$Y_i=\sqrt{B_{i1}^2+B_{i2}^2}\,S_{di} \tag{10}$$

If the modal shapes of the structure are normalized in such a way that:

$$[\phi]^T[M][\phi]=[I] \tag{11}$$

where [I] is a 3nx3n identity matrix, Eq.10 can be expressed as:

$$Y_i=\sqrt{M_{xxi}+M_{yyi}}\,S_{di} \tag{12}$$

where M_{xxi} and M_{yyi} are the equivalent ith-mode translation masses in the X and Y directions respectively. The concept of equivalent mass in a given direction corresponds to a coefficient, which multiplied by the acceleration spectrum value related to ith-mode, determines the base shear for the ith-mode of the structure in the given direction.

Therefore the maximum contribution of the ith-mode to the relative displacement response of the structure, according to Eq.2 is:

$$\{U\}_i=\{\phi\}_i Y_i \tag{13}$$

Replacing Eq.12 into Eq.13 yields:

$$\{U\}_i=\{\phi\}_i\sqrt{M_{xxi}+M_{yyi}}\,S_{di} \tag{14}$$

Eq.14 corresponds to the maximum contribution from the ith-mode to the relative displacement response of the structure in the time domain, as well as the direction of incidence of the ground acceleration causing this maximum.

The maximum forces caused by the ith-mode maximum displacements obtained from Eq.14 are given by:

$$\{F\}_i=[M]\{\phi\}_i\sqrt{M_{xxi}+M_{yyi}}\,S_{ai} \tag{15}$$

where S_{ai} is the value of the acceleration spectrum related to ith-mode. With the maximum modal forces (Eq. 15) the maximum contribution of each mode can be determined for shear stress, overturning moment and torque occurring on each floor of the structure.

Finally, the global response of the structure (relative displacements and stresses) are obtained by combining the maximum contribution of each mode by an appropriate modal superposition criterium.

5 APPLICATION OF THE METHOD

Three buildings were analyzed. One of them have modal coupling and a small static eccentricity. The other two have their normal mode uncoupled, one with a large and the other with a small static eccentricity.

Each building was subjected to the following:

-Actual ground acceleration data from El Centro 40 NS and LLolleo 85 N10E.

-Acceleration response spectra from the actual ground acceleration data from El Centro 40 NS and LLolleo 85 N10E.

-Acceleration design spectra from the Chilean Code for Earthquake Resistant Design of Buildings (INN 1972).

-Acceleration design spectra from the U.S. Uniform Building Code.

-Acceleration design spectra from the Mexican Design Code.

Acceleration response spectra and design spectra were utilized in the application of the proposed method to obtain the maximum response for each building as indicated below.

Maximum displacement response for each mode was obtained using Eq.14. Maximum modal forces necessary to determine maximum modal stresses, were determine using Eq.15. The maximum global response for each building was obtained using modal combination criteria i.e., square root of sum of square (SRSS), double sum complete (DSC), combination cuadratic complete (CQC) and the criteria of the Chilean Code for Earthquake Resistant Design of Buildings (NCh)

In order to have a basis of comparison the maximum response of the structure were also determined using a time step integration procedure (actual earthquake data) and the standard spectral method. This was carried out considering different directions of incidence of the ground acceleration (between 0° and 180°).

In the case of utilizing response spectra or design spectra, the structure displacements and stresses were obtained by standard procedure using the modal combination criteria (SRSS, DSC, CQC, and NCh)

6 ANALYSIS OF RESULTS

A comparative analysis of the results obtained above was carried out. The displacements of the centers of mass and the edges of the top deck were compared. The basal stresses obtained in the three buildings were also compared.

The results are shown in tables 1 to 4 for the three buildings analyzed.

Table 1 shows average differences between the maximum responses obtained by time step integration procedure and those obtained with the proposed method using the above mentioned modal combination criteria. Average differences values below 14% can be observed when modal combination criteria (SRSS, DSC, and CQC) are utilized.

Table 2 shows maximum differences between the maximum response obtained by time step integration procedure and those obtained with the proposed method for each modal combination criteria. It can be seen that the modal combination criterium SRSS gives minimum dispersion.

Table 3 shows average differences between the maximum displacements and stresses obtained using the spectral method and those obtained with the proposed method. Average difference values below 20% can be observed when modal combination criteria SRSS, DSC, and CQC are utilized.

Table 4 shows maximum differences between the maximum displacements and stresses obtained using the spectral method and those obtained with the proposed method for each modal combination criterium. It can be observed that the differences are always positive when the modal combination criterium SRSS is used, thus resulting in safer design. However, when the modal combination criterium prescribed by the chilean code is utilized (see Tables 2 and 4), larger differences can be seen which results in over design.

Table 1. Average differences (%) between the proposed method and the results obtained from the time step integration procedure.

Structural	Superposition Criteria			
Response	SRSS	DSC	CQC	NCh
Displacements	9.7	8.9	8.8	44.2
Rotations	9.8	2.9	3.1	36.7
Edges Displ.	14.0	10.5	10.5	48.6
Base Shear	3.4	2.1	2.1	42.5
Base Torque	3.8	-3.0	-2.8	40.4

Table 2. Maximum differences (%) between the proposed method and the results obtained from the time step integration procedure.

Maximum	Superposition Criteria			
Differences	SRSS	DSC	CQC	NCh
Positive	30.2	37.3	37.3	73.1
Negative	10.2	55.8	55.4	-

Table 3. Average differences (%) between the proposed method and the results obtained from standard spectral method.

Structural	Superposition Criteria			
Response	SRSS	DSC	CQC	NCh
Displacements	19.7	9.8	9.9	30.9
Rotations	1.2	3.0	3.0	3.6
Edges Displ.	17.2	12.6	12.6	27.7
Base Shear	14.5	4.5	4.7	29.7
Base Torque	1.0	2.5	2.5	3.5

Table 4. Maximum differences (%) between the proposed method and the results obtained from standard spectral method.

Maximum	Superposition Criteria			
Differences	SRSS	DSC	CQC	NCh
Positive	30.6	41.5	41.4	40.3
Negative	-	52.6	52.2	-

7 CONCLUSIONS

The main advantage of the proposed method is that it avoids the use of arbitrary two-orthogonal directions on seismic analysis to obtain the maximum design stresses.

In the proposed method it is not necessary to carry out the computation of the orthogonal directional effects.

The proposed method estimates very well the seismic responses of the structure when the square root of sum of squares (SRSS) modal combination technique is utilized.

The proposed method is more precise and safer than conventional seismic design methods.

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