UNIVERSIDAD AUTÓNOMA DEL ESTADO DE MÉXICO

FACULTAD DE INGENIERÍA

ANÁLISIS COMPARATIVO ENTRE UN PROCEDIMIENTO PARA ESTIMAR EL EFECTO DE LA TORSIÓN ACCIDENTAL Y SIMULACIONES MONTE CARLO, EN EL MARCO DEL ANÁLISIS PASO A PASO NO LINEAL DE EDIFICIOS

T E S I S

QUE PARA OBTENER EL TÍTULO DE

MAESTRO EN CIENCIAS DE LA INGENIERÍA

P R E S E N T A

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Contenido

Resumen

Se presenta un análisis comparativo entre las respuestas obtenidas con dos procedimientos que consideran torsión sísmica accidental en análisis dinámico no lineal de edificios: 1) a partir de valores típicos de excentricidades accidentales recomendados en códigos de construcción y diseño y, 2) mediante la variación de parámetros que inciden en la torsión accidental (masas y rigideces) y simulaciones Monte Carlo. La comparación se realiza a partir de valores de respuestas medidos en términos de demandas de ductilidad y distorsiones de entrepiso. En el estudio se consideran dos modelos de edificios simétricos de varios niveles. Los resultados muestran que considerar el efecto de la torsión accidental a partir de valores típicos de excentricidades accidentales conduce a demandas de ductilidad mayores que las obtenidas de las simulaciones Monte Carlo.

Abstract

A comparative analysis between the responses obtained with two procedures that consider the seismic accidental torsion on the dynamic nonlinear analysis of buildings is presented: 1) from typical values of accidental eccentricities recommended in design-building codes and, 2) by means the variation of parameters that incise in the accidental torsion (masses and stiffness) and Monte Carlo simulations. The comparison is performed from response values measured in terms of ductility demands and lateral distortions. In the study, two models of multi-level symmetrical buildings are considered. Results show that to consider the effect of accidental torsion with typical accidental eccentricity values leads to ductility demands higher than those obtained with Monte Carlo simulations.

Presentación

En el capítulo 1 se presenta el protocolo de tesis con número de registro MSCING-0719 aprobado por la Comisión de Revisión de Protocolo. En el capítulo 2 se presenta el artículo que corresponde al trabajo de investigación que se desarrolló durante la estancia en el Programa de Maestría de la Universidad Autónoma del Estado de México. El artículo que lleva por nombre "Accidental torsion within the frame of nonlinear dynamic analysis using code accidental eccentricities and Monte Carlo simulations" se envió a la revista *Engineering Structures* para su revisión y publicación. En él se detallan los aspectos principales de la investigación, los resultados y las conclusiones obtenidas.

Capítulo 1. Protocolo de tesis

PROTOCOLO

DE

TESIS

Universidad Autónoma del Estado de México

Facultad de Ingeniería

Programa de Maestría en Ciencias de la Ingeniería

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Información del protocolo

Título

Análisis comparativo entre el método de análisis paso a paso no lineal y (un método que recurra a técnicas de simulación) incorporando el efecto de la torsión sísmica accidental en edificios.

Asesor

Dr. Jaime De la Colina Martínez

Área académica

Área de Estructuras

Planteamiento del tema de investigación

Introducción

Uno de los objetivos del diseño sísmico es proporcionar a la estructura la resistencia necesaria para soportar sismos moderados y sismos de alta magnitud con una probabilidad mínima de suceder. En el Reglamento de Construcciones de la CD MX y sus Normas Técnicas Complementarias para Diseño por Sismo se establecen métodos y parámetros de análisis a fin de cumplir con tal objetivo.

De manera general el problema que representa el análisis sísmico de una estructura se reduce a calcular los desplazamientos que se producen en ésta debido al movimiento generado en su base por una acción sísmica determinada. Tales desplazamientos se obtienen al resolver la o las ecuaciones de movimiento asociadas al sistema estructural. A su vez éstos permiten calcular las fuerzas actuantes y los elementos mecánicos en los diferentes elementos de la estructura.

Para sistemas de uno o varios grados de libertad la ecuación dinámica de movimiento puede expresarse como (Chopra, 2012):

$$\mathbf{M}\ddot{u} + \mathbf{C}\dot{u} + \mathbf{K}u = p(t)$$

o bien:

$$\mathbf{M}\ddot{u} + \mathbf{C}\dot{u} + \mathbf{K}u = -\mathbf{M}\ddot{u}(t)$$

en donde $\mathbf{M}$, $\mathbf{C}$ y $\mathbf{K}$ son las matrices de masa, amortiguamiento y rigidez; respectivamente. A su vez u, $\dot{u}$ y $\ddot{u}$ son los vectores de desplazamientos, velocidades y aceleraciones. Cuando no es posible obtener una solución cerrada en forma analítica de la ecuación de movimiento resulta necesario recurrir a métodos aproximados. Esto ocurre generalmente cuando la fuerza excitadora varía arbitrariamente con el tiempo, o bien, cuando el sistema no es lineal (Chopra, 2012). Para este último caso, la ecuación que rige el equilibrio dinámico está dada por:

$$\mathbf{M}\ddot{u} + \mathbf{C}\dot{u} + \mathbf{F}(u) = p(t)$$

o bien:

$$\mathbf{M}\ddot{u} + \mathbf{C}\dot{u} + \mathbf{F}(u) = -\mathbf{M}\ddot{u}(t)$$

Aquí, $\mathbf{F}(u)$ es la matriz de fuerzas de restitución, las cuales van cambiando conforme al desplazamiento del sistema en cada instante de tiempo. La solución de este último conjunto de ecuaciones diferenciales puede obtenerse mediante procedimientos numéricos que consideran el comportamiento lineal del sistema durante intervalos pequeños de carga (Meli, 2016).

Métodos de análisis sísmico

Hoy en día se puede estudiar el comportamiento de una estructura sujeta a la acción sísmica recurriendo a dos tipos de análisis, ya sea mediante un análisis estático o bien, mediante uno dinámico. En un análisis estático, la solución que se obtiene es independiente del tiempo, mientras que, en uno dinámico, ésta consiste en la solución para todos los instantes de tiempo dentro del periodo de estudio. Este último tipo de análisis representa de una manera más precisa el comportamiento de la estructura durante todo el evento sísmico.

Como se comentó anteriormente, en los códigos de diseño (*e.g.*, RCDF, 2004; UBC, 1994) se especifican diversos procedimientos para el análisis sísmico de estructuras. El método de análisis estático es uno de ellos y es aplicable a estructuras que se ajustan a ciertos tipos de estructuración y alturas determinadas. Básicamente, el análisis estático consiste en someter a la estructura ante un conjunto de fuerzas laterales (horizontales) equivalentes a la acción sísmica. Para este caso dicha acción está definida mediante una fuerza asociada a la demanda espectral correspondiente, según el periodo fundamental de la estructura.

Con base en las características de la estructura que se analice (estructuración, zonificación, altura entre otros), si no se cumplen con los requisitos establecidos para realizar un análisis sísmico estático, es necesario aplicar algún otro procedimiento. En el RCDF y sus Normas Técnicas Complementarias para Diseño por Sismo se presentan dos métodos dinámicos para el análisis sísmico de estructuras: 1) el análisis modal espectral; y 2) el análisis paso a paso. A diferencia del método de análisis estático, ambos métodos pueden emplearse para el análisis sísmico de cualquier estructura. No obstante, una de las diferencias entre el método de análisis modal espectral y el método paso a paso reside en la forma en cómo se define la excitación sísmica de diseño. En el análisis modal generalmente se recurre a espectros de diseño para el cálculo de la respuesta de la estructura, mientras que, en el método de análisis paso a paso la excitación se representa mediante acelerogramas de temblores reales o de movimientos simulados, o bien, mediante combinaciones de ambos, en vez de espectros. En relación a esto último el RCDF y sus Normas Técnicas Complementarias para Diseño por Sismo establecen como una limitación utilizar no menos de cuatro movimientos representativos para tal efecto (además de algunos otros requisitos adicionales).

En cuanto a la solución, para el análisis modal se recurre a la solución de las ecuaciones de equilibrio dinámico a partir de los modos de vibración o formas modales del sistema. La respuesta de cada modo vibración puede calcularse en forma independiente de las otras a partir de espectros de diseño estipulados en algunos códigos de diseño. Cada forma modal responde con su propio patrón particular de deformación y al combinarse determinan la respuesta total del sistema estructural. Podemos definir a un espectro de respuesta como una representación gráfica de la respuesta máxima (expresada en términos de desplazamiento, velocidad, aceleración o algún otro parámetro de interés) que produce una acción dinámica determinada en una estructura u oscilador de un grado de libertad (Granados, 2013). A su vez, el espectro de diseño (el cual se basa en el análisis estadístico de un conjunto de espectros de respuesta) se trata, en un sentido general, de una gráfica representativa de los movimientos del terreno registrados en el sitio durante sismos pasados. El análisis paso a paso requiere de la solución las ecuaciones de movimiento en cada instante de tiempo para una determinada excitación sísmica correspondiente a un acelerograma. Algunos tipos de procedimientos paso a paso en el tiempo lo son: a) los métodos basados en la interpolación de la función de la excitación, b) los métodos basados en expresiones de diferencias finitas de la velocidad y aceleración, y c) los métodos basados en la variación supuesta de la aceleración. En relación a este último se puede referir el método β de Newmark (*e.g*. Chopra, 2012) o bien, el método Runge-Kutta (*e.g*., Hidalgo y Ruiz, 2010; Sánchez, 2002).

Aun cuando ambos métodos son aplicables, de acuerdo con Torre E. (2006) estos procedimientos de análisis pueden ser insuficientes para describir el comportamiento real de las estructuras, ya que para ciertas condiciones éstas pueden incursionar en el intervalo no lineal. Lo anterior sugiere que los métodos empleados comúnmente para el estudio de la respuesta lineal de una estructura sujeta a la acción sísmica, no son adecuados para estudiar su comportamiento inelástico (no lineal), tal es el caso del método de análisis modal espectral.

Planteamiento del problema

Para estudiar de manera más realista la respuesta de una estructura sujeta a la acción sísmica, resulta conveniente realizar un análisis de su comportamiento inelástico el cual generalmente se presenta para sismos intensos. En este sentido, el método de análisis paso a paso puede implementarse tanto para estructuras lineales y no lineales.

El Reglamento de Construcciones de la CD MX establece que para edificaciones que exceden ciertos límites de altura (entre 80 y 120 metros) debe verificarse el diseño estructural con un análisis dinámico no lineal paso a paso. Lo anterior con el objetivo de revisar que el diseño propuesto cumpla con lo especificado en dicha norma.

El método de análisis paso a paso presenta algunas particularidades referentes a su aplicación. Una de ellas consiste en la selección de acelerogramas cuya intensidad de diseño sea compatible con la establecida en los códigos de diseño. Otra reside en el problema que implica la obtención de acelerogramas (registrados o sintéticos) que representen las características del movimiento del sitio de interés.

Otro aspecto que se debe tomar en cuenta (de acuerdo al reglamento vigente) son las incertidumbres que existen en cuanto a los parámetros que definen la no linealidad de las estructuras. Dos parámetros asociados a dichas incertidumbres son: la rigidez y resistencia de los elementos resistentes a carga lateral y las características como la magnitud y posición de las masas. Estos parámetros conducen a la excentricidad accidental que junto con la excentricidad propia del modelo de análisis (comúnmente referida como excentricidad natural) definen la excentricidad total del modelo.

En este punto conviene señalar que en la nueva normativa emitida no se ofrecen procedimientos o directrices claras para incorporar el efecto de la excentricidad accidental en el análisis sísmico dinámico paso a paso de las estructuras, particularmente en el caso de edificios altos. De lo anterior es claro que hace falta definir para la revisión del diseño, además de las características de los acelerogramas, definir las características del modelo para que se incluyan de manera adecuada la excentricidad accidental en un análisis paso a paso.

Finalmente, es importante destacar la importancia que tienen los requisitos adicionales que especifican los principales códigos de diseño en relación al uso y aplicabilidad del método paso a paso en el tiempo, que en un sentido general, precisan su uso frente a métodos de análisis que son más prácticos que recurren a simplificaciones para considerar el comportamiento inelástico del sistema estructural (*e.g.*, Donobhan, 2007).

Lo anterior, aunado a la poca disponibilidad y a los escasos trabajos referentes a problemas de este tipo resultan en la necesidad de mostrar cómo abordarlos, preferentemente a partir de procedimientos simplificados que posean un nivel de aproximación suficiente. Queda claro que el desarrollo de un trabajo que muestre de manera clara y específica los principales criterios a considerar en el proceso de análisis (apegado a un código de diseño determinado) y/o el establecimiento de recomendaciones de análisis, son una alternativa para entender de una manera más clara el proceso que éste implica.

Objetivo general

Componente no lineal: Comparar las respuestas obtenidas de un análisis paso a paso no lineal que incluya recomendaciones para tomar en cuenta la torsión accidental, con las respuestas obtenidas de simulaciones que consideren la variación de los parámetros (masas y rigideces) que inciden en la excentricidad accidental. De manera particular, la comparación se hará en términos de demandas de ductilidad globales.

Componente lineal: Se pretende comparar las respuestas obtenidas de un análisis lineal paso a paso que incluya recomendaciones para tomar en cuenta la torsión accidental de acuerdo a la nueva normatividad con las respuestas obtenidas de un análisis modal espectral del mismo reglamento.

Hipótesis

La respuesta sísmica calculada mediante un análisis paso a paso no lineal, incluyendo el efecto de la excentricidad accidental, no excede en más de un 10% de la respuesta correspondiente calculada a partir de un análisis (no lineal) que recurra a técnicas de simulación.

Metodología

I. Revisión del estado del arte y acopio de información.

II. Revisión de los principales códigos de diseño en relación al uso del procedimiento de análisis sísmico dinámico paso a paso.

III. Generación de modelos representativos para ser estudiados.

IV. Revisión del efecto de cada uno de los parámetros y variables que intervienen en el proceso de análisis, y por tanto, en la respuesta de los modelos propuestos.

V. Establecimiento de la acción sísmica a partir de un conjunto de sismos (considerándolos como un proceso estocástico).

VI. Cálculo de la respuesta de los modelos propuestos a partir de los métodos de análisis modal y paso a paso incluyendo los efectos de la excentricidad accidental de acuerdo a la normativa vigente.

VII. Cálculo de la respuesta de los modelos propuestos a partir de un método que recurra a técnicas de simulación no lineal y el método de análisis paso a paso no lineal incluyendo los efectos de la excentricidad accidental.

VIII. Análisis y estudio de la respuesta de los modelos abordados.

IX. Establecimiento de conclusiones y recomendaciones referentes al uso del método de análisis sísmico paso a paso en el diseño sísmico de edificios.

X. Elaboración del trabajo escrito.

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Cronograma de actividades

	Actividades	Trimestre 1°	2°	3°	4°	5°	6°	7°	8°
I.	Revisión del estado del arte y acopio de información.	x	x	x	x	x			
II.	Revisión de los principales códigos de diseño en relación al uso del procedimiento de análisis sísmico dinámico modal y paso a paso.		x						
III.	Generación de modelos representativos para ser estudiados a partir de un análisis símico dinámico modal y un análisis paso a paso.			x					
IV.	Revisión del efecto de cada uno de los parámetros y variables que intervienen en el proceso de análisis, y por tanto, en la respuesta de los modelos propuestos.			x					
V.	Establecimiento de la acción sísmica a partir de un conjunto de sismos (considerándolos como un proceso estocástico).			x	x				
VI.	Cálculo de la respuesta de los modelos propuestos a partir de los métodos de análisis modal y paso a paso.					x	x		
VII.	Análisis y estudio de la respuesta de los modelos abordados.					x	x		
VIII.	Establecimiento de conclusiones y recomendaciones referentes al uso del método de análisis sísmico paso a paso en el diseño sísmico de edificios.						x		
IX.	Elaboración del trabajo escrito.						x	x	x

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Capítulo 2. Artículo de investigación

Manuscript Details

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Title	ACCIDENTAL TORSION WITHIN THE FRAME OF NONLINEAR DYNAMIC ANALYSIS USING CODE ACCIDENTAL ECCENTRICITIES AND MONTE CARLO SIMULATIONS
Article type	Research Paper

Abstract

A comparative analysis between the responses obtained with two procedures that consider the accidental torsion on the dynamic nonlinear analysis of buildings is presented. The first one involves the use of typical values of accidental eccentricities used in the design process. The second procedure involves the variation of parameters that incise in the accidental torsion by means Monte Carlo simulations. The comparison is performed from the responses obtained, measured in terms of concentrated inelastic deformations and lateral distortions. In the study, two models of multi-level symmetrical buildings are considered. The CANNY-E program is used for the nonlinear dynamic analysis of the proposed structural models. The results suggest moving the position of the center of mass a distance equal to 0.05b to consider the effect of accidental torsion on the dynamic nonlinear analysis of buildings.

Keywords	Accidental torsion; accidental eccentricity; Monte Carlo simulations; nonlinear analysis;
Taxonomy	Earthquake Engineering, Seismic Structural Response Analysis, Engineering Structure, Torsional Loads
Manuscript region of origin	North America

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January 24, 2020.

Dr. P.L. Gould

Editor-in-Chief Engineering Structures

Dear Dr. Gould,

We are sending to you the manuscript entitled: “Accidental torsion within the frame of nonlinear dynamic analysis using code accidental eccentricities and Monte Carlo simulations” which we are submitting for exclusive consideration of publication as a research paper in Engineering Structures.

The paper shows a comparative analysis between the responses obtained with two procedures to consider the accidental torsion on the dynamic nonlinear analysis. The first one corresponds to a procedure in which the effect of accidental torsion is estimated from typical values of accidental eccentricities used in design-buildings codes. The second corresponds to a procedure in which the effect of accidental torsion is included by simulation techniques. The study employs buildings models of 4 and 8 levels, designed in accordance with different levels of accidental torsion. In the simulation process, uncertainties associated with the rigidity of the structural elements, as well as the magnitude and position of the masses are considered. Results show that to consider the effect of accidental torsion with typical accidental eccentricity values (proposed in some building codes) for nonlinear analysis purposes, leads to ductility demands higher than those obtained with Monte Carlo simulations.

Thank you for your consideration. Sincerely, the authors:

Francisco Manzanarez Morones

Jaime De la Colina Martínez

Jesús Valdés González

ACCIDENTAL TORSION WITHIN THE FRAME OF NONLINEAR DYNAMIC ANALYSIS USING CODE ACCIDENTAL ECCENTRICITIES AND MONTE CARLO SIMULATIONS

Francisco Manzanarez Morones, Jaime De la Colina Martínez, Jesús Valdés González

Highlights

- Ductility demands for building models designed with three different distributions of accidental eccentricities are practically equal each other.
- Differences on ductility demands of up to 65% were observed between two approaches used to include accidental torsion with accidental eccentricities (the first with torsional moments applied at the slabs and the second with eccentric masses at each floor).
- When the accidental torsion is included with eccentric masses at each floor the ductility demands are similar to those computed with the Monte Carlo simulations.
- The results corresponding to the approach that uses torsional moments applied at the slabs are not consistent with the results obtained from Monte Carlo simulations.
- For analysis purposes, the use of mass eccentricities equal to 0.05*b* (where *b* is the dimension in plan of the building, perpendicular to the direction of analysis) leads to ductility demands that are almost equal (but larger than) the ductility demands computed with the Monte Carlo method.

ACCIDENTAL TORSION WITHIN THE FRAME OF NONLINEAR DYNAMIC ANALYSIS USING CODE ACCIDENTAL ECCENTRICITIES AND MONTE CARLO SIMULATIONS

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Abstract

A comparative analysis between the responses obtained with two procedures that consider the accidental torsion on the dynamic nonlinear analysis of buildings is presented. The first one involves the use of typical values of accidental eccentricities used in design process. The second procedure involve the variation of parameters that incise in the accidental torsion by means Monte Carlo simulations. The comparison is performed from the responses obtained, measured in terms of concentrated inelastic deformations and lateral distortions. In the study two models of multi-level symmetrical buildings are considered. The CANNY-E program is used for the nonlinear dynamic analysis of the proposed structural models. The results suggest move the position of the center of mass a distance equal to 0.05b to consider the effect of accidental torsion on the dynamic nonlinear analysis of buildings.

Keywords: accidental torsion; accidental eccentricity; Monte Carlo simulations; nonlinear analysis.

1. Introduction

Currently, some building codes (*e. g.,* [1, 2]) demand the revision of the structural design of certain edifications with a dynamic step-by-step nonlinear analysis. This provision is included in the Mexico City building code RC-CDMX [1] for buildings that exceed the height limits defined in Table 1. This provision is required to verify that the structural design comply with nonlinear capacities demanded in strong seismic motions.

While this type of analysis is intended for a *realisti*c assessment of a structural system behavior (mainly in the nonlinear range), its application poses some *difficulties*. One of them is the selection of accelerograms with similar intensities to the design earthquake (defined by the spectrum design). Another is the selection of accelerograms (real or artificial) with frequency contents and durations similar to the expected seismic motions at a given site.

Table 1
Height limits above which a dynamic step-by-step nonlinear analysis is required [1].

Geotechnical zones	Structural configuration	Height in m
Zone II (transition) and III (lake)	Regular	120
	Irregular	100
	Very irregular	80

An additional difficulty in applying the dynamic nonlinear analysis is the inclusion of accidental torsion. The latter, together with the natural torsion, define the total torsion that the structural system must resist. The phenomenon of seismic torsion is addressed by building codes with simplified procedures intended to include its effects on the analysis. Such procedures involve the use of design eccentricities e_d.

Addition to a natural eccentricity e_n (which considers the torsion inherent of the structural system) the design eccentricity e_d includes an accidental eccentricity e_a to estimate the effects of accidental torsion.

The latter is due to the uncertainty (regarding theoretical or nominal values) of parameters associated with the masses and rigidities of the structural system, mainly. Natural eccentricity e_n is determined based on the distribution of masses and rigidities in the structural system and is calculated with theoretical or nominal values. As for the accidental eccentricity e_a, values are recommended in most building codes. The Uniform Building Code [3] specifies a value equal to 0.05 times the dimension in plan b of the building, perpendicular to the direction of analysis. On the other hand, the National Building Code of Canada [4] specifies a value equal to $0.1b$. These and some other values are presented in Table 2.

Table 2
Accidental eccentricity values e_a established in several codes.

Code/Standard	e_a / b
Uniform Building Code [3]	0.05
National Building Code of Canada [4]	0.10
Eurocode [5]	0.05
Building Code for the Federal District [6]	0.10
New Zealand [7]	0.10

In the new publication of the RC-CDMX [1], the proposed procedure to estimate the effect of accidental torsion on the design process is different from that exposed in previous editions (*e. g.*, [6]). The new procedure is similar to established in the ASCE/SEI 7 [8] *standard* for the case of static analysis, in which the effect of accidental torsion is estimated from eccentricities vary with height in accordance with $A_x{\cdot}0.05b$, where factor A_x is limited to a maximum value of three. The above leads to accidental eccentricity values ranging from $0.05b$ to $0.15b$. For the dynamic case, the code sets a value equal to $0.05b$, constant for the entire building height.

Thus, there is a variation of the accidental eccentricity values e_a specified in building codes, which have been the result of previous studies [9]. Initially, many of the research associated with the seismic torsion phenomenon was based on highly idealized simplified models, which limited in some way the generalization of the results obtained (*e. g.*, [10]). One of the limitations was the consideration of single-floor models, which, according to some authors, were not suitable for assessing the phenomenon of seismic torsion in multi-story models. The results obtained from these researches provided a basis for subsequent studies where increasingly *refined* models were used. Some were based on the study of models with elastic behavior (*e. g.*, [11]), however, the results associated with such studies were questioned, mainly due to the incursion into the nonlinear interval during high intensity earthquakes. This led to the study of the phenomenon in the inelastic interval (*e. g.*, [12]). Recent studies have been based on the evaluation of the established procedures in the building and design codes associated with the effect of accidental seismic torsion [*e. g.*, [13, 14]). However, some or others have focused on the study of the phenomenon of accidental torsion with a probabilistic approach [*e. g.*, [15]), which, given the inconsistencies of the pertinent recommendations, seems a more appropriate procedure.

In this work, a comparative analysis between the responses obtained with two procedures to consider the accidental torsion on the dynamic nonlinear analysis is presented. The first one corresponds to a deterministic procedure in which the effect of accidental torsion is included with typical values of accidental eccentricities used for design. The second procedure corresponds to the application of the Monte Carlo method [16] in which the effect of accidental torsion is included with simulation techniques. The study employs building models of 4 and 8 levels, *designed* to resist different levels of accidental torsion. In the simulation process, uncertainties associated with the rigidity of the structural elements, as well as the magnitude and position of the masses are considered. In addition to the lateral load effect (due to the action of several seismic motions), the effect of gravitational loading is included.

The objective of the study is to know the differences of ductility demands and interstory distortions computed with both procedures. These two parameters are used in structural review processes, in which the safety of the buildings is evaluated with dynamic nonlinear analyses. In this study, the use of the dynamic nonlinear analyses is oriented to the evaluation of the effects of accidental torsion and not to the structural review process by itself. Its application (particularly in the nonlinear range) requires both the selection and the scaling criteria of ground motions. Recommendations to perform structural revisions are found in the ASCE/SEI 7 [8] *standard* and in the RC-CDMX [1]. While both criteria are similar, the latter code is used here as the basis for the comparison of the procedures to include accidental torsion as commented above within the dynamic nonlinear analysis.

2. Structural models

In this study two steel structural models are used. The first one is referred as E4 and represents a generic building of 4 levels with dimensions in plan of 12.0m by 12.0m and with interstory heights of 3.2m. The second model is referred as E8 and represents a generic building of 8 levels with plan dimensions of 18.0m by 18.0m and with interstory heights of 3.2m.

The structural system adopted to support both gravitational and lateral loads, consists of steel rigid frames arranged in two orthogonal directions. The model E4 has three frames along each orthogonal direction, while the model E8 has four frames. Figures 1 and 2 show the geometry of models E4 and E8, respectively. It is observed that both structural models correspond to regular three-dimensional structures.

For design purposes, the response of both structural models was estimated with a spectral modal analysis. The design seismic actions were defined with design spectra, which were affected by two factors: seismic behavior R and over-resistance Ω_0. Floor systems were modeled as rigid diaphragm. The design included both the effect of accidental torsion and the bidirectional effects. For the design of the structural models, the criterion of load and resistance factor design (LRFD) was followed, according with the RC-CDMX [1]. In general, the steel norms of this code lead to designs that result similar to the obtained with the ANSI/AISC 360 specifications [17].

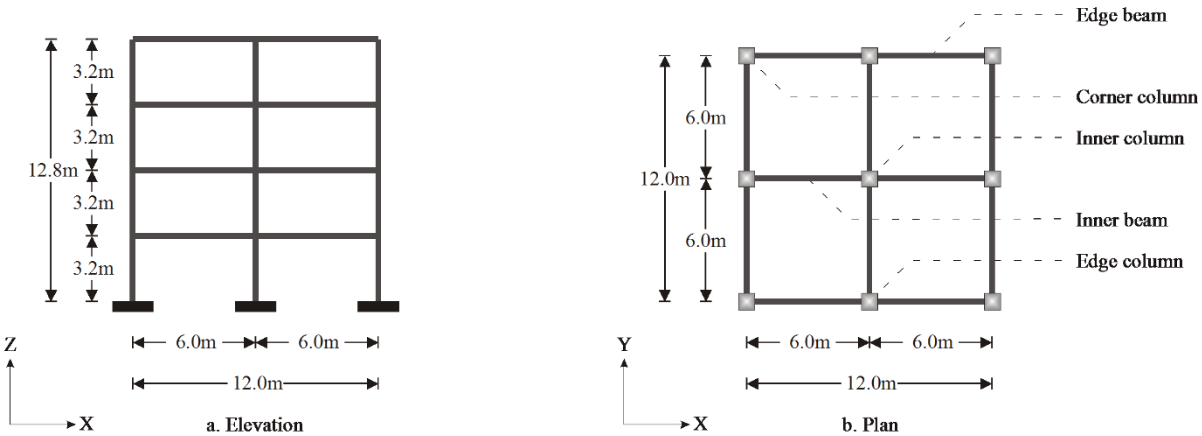


Figure 1. Structural model E4.

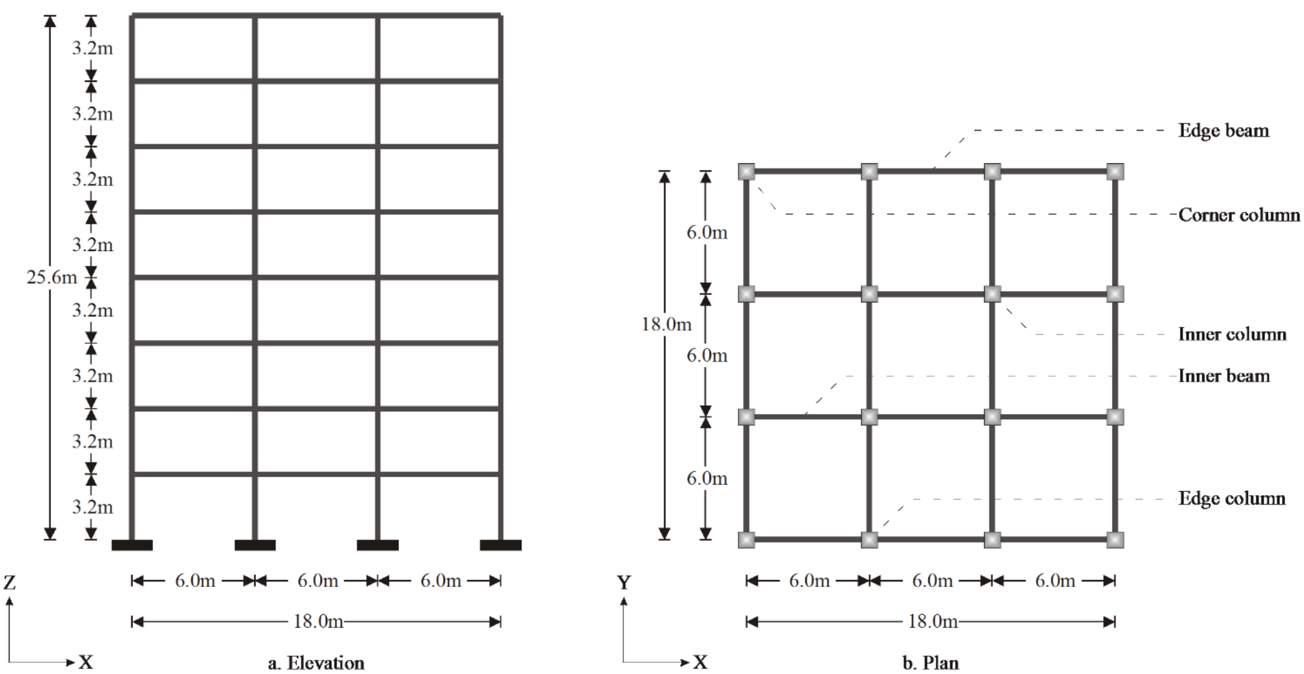


Figure 2. Structural model E8.

For all frames, W and HSS sections are used for beams and columns, respectively. Member sections of models E4 and E8 are summarized in Tables 3 and 4, respectively. In both tables, three types of columns (inner, edge and corner) and two types of beams (inner and edge) are identified. These models do not correspond to actual buildings; they are assumed as generic models representing well-structured buildings with uniform distributions of masses and stiffness. Translational (*T1* and *T2*) and rotational (*T3*) vibration periods are equal to 1.0 and 0.7 seconds (respectively) for model E4. As for the model E8, the corresponding periods resulted equal to 2.0 and 1.5 seconds. These values are identified in Figure 3, along with the response spectra of the ground motions used in the dynamic nonlinear analysis.

Table 3
Steel sections, E4 model.

Element	Interstory / Level	Section
Inner columns	1 and 2 3 and 4	HSS14" x 14" x 5/8" HSS12" x 12" x 1/2"
Edge columns	1 and 2 3 and 4	HSS14" x 14" x 1/2" HSS12" x 12" x 3/8"
Corner columns	1 and 2 3 and 4	HSS14" x 14" x 3/8" HSS12" x 12" x 1/4"
Inner beams	1 to 3 4	W 12" x 26 lb/ft W 12" x 22 lb/ft
Edge beams	1 to 4	W 12" x 22 lb/ft

Table 4
Steel sections, E8 model.

Element	Interstory / Level	Section
Inner columns	1 and 2 3, 4 and 5 6, 7 and 8	HSS16" x 16" x 5/8" HSS14" x 14" x 1/2" HSS 12" x 12" x 3/8"
Edge columns	1 and 2 3, 4 and 5 6, 7 and 8	HSS16" x 16" x 1/2" HSS14" x 14" x 3/8" HSS 12" x 12" x 5/16"
Corner columns	1 and 2 3, 4 and 5 6, 7 and 8	HSS16" x 16" x 5/8" HSS 14" x 14" x 5/16" HSS 12" x 12" x 1/4"
Inner beams	1 to 7 8	W 12" x 26 lb/ft W 12" x 22 lb/ft
Edge beams	1 to 6 7 and 8	W 12" x 26 lb/ft W 12" x 22 lb/ft

3. Ground motions

3.1 Ground motions for nonlinear analysis

The application of the dynamic nonlinear structural revision procedure requires the selection and scaling of ground motions. These selection and scaling procedures are usually included in the building codes. Here, the criteria specified in the Mexico City building code [1] are described by completeness. Although some of the established criteria for selecting and scaling ground motions differ from one code to another, in general, they aim to establish guidelines for the selection of a prescribed number of ground motions representative of the expected seismic motions at given site.

3.2 Selection of ground motions

For the purpose of using nonlinear dynamic analyses, sets of seismic excitations are selected as indicated in section 6.2.1 of the Complementary Technical Standards for Seismic Design (NTC-DS) of the RC-CDMX [1]. Each excitation consists of a pair of accelerograms corresponding to two horizontal orthogonal components *C1* and *C2* of the ground motion.

The regulation establishes the use of accelerograms coming from either real and/or simulated ground motions. In accordance with the code, seismic excitations must be independent of each other and have intensities, durations, and frequencies contents similar to those observed in real earthquake records with intensities equal to those assumed in design seismic actions. As for the number of ground motions, this is defined in function of the vibration period of the site T_S. The code prescribes the use of at least 8 ground motions if T_S is less than 2 seconds. For T_S equal to or greater than 2 seconds, no less than 12 ground motions should be used.

For the case of E4 model, the seismic excitations were obtained from accelerograms coming from real earthquakes recorded in the CU station of the UNAM Engineering Institute. General information of these excitations are shows in Table 5. For the case of E8 model, seismic excitations (Table 6) were selected from stations whose records have frequency contents similar to those recorded at the site of interest. In both cases, the design spectrum was taken as a reference for the selection of ground motions.

Table 5
Seismic excitations used in the dynamic nonlinear analysis of the E4 model.

Id	Earthquake	Location	Date	Station	V_{S30} [a] in m/s	Type of soil [b]
E1-SH	Mexico 1985	Michoacán, Mexico	19/09/1985			
E2-SH		Guerrero, Mexico	25/04/1989			
E3-SH		Colima, Mexico	09/10/1995			
E4-SH			22/11/2005	Engineering Institute, UNAM	367.31	C
E5-SH		Oaxaca, Mexico	28/09/2010			
E6-SH		Chiapas, Mexico	10/12/2015			
E7-SH			04/02/2016			
E8-SH		Oaxaca, Mexico	27/06/2016			

[a] Velocity of propagation of shear waves measured up to 30 meters deep from the surface. Data obtained from NEHRP [18].
[b] Classification of soil type: very dense soil and / or soft rock (C). Data obtained from Table 7.

Table 6
Seismic excitations used in the dynamic nonlinear analysis of the E8 model.

Id	Earthquake	Location	Date	Station	V_{S30} [a] in m/s	Type of soil [b]
E1-SS	Northridge	California, USA	17/01/1994	Sylmar	325.07	D
E2-SS	Azmit	Azmit, Turkey	17/08/1999	Ambarli-Termik Santrali	246.69	D
E3-SS	Kobe	Kobe, Japan	16/01/1995	Port Island	201.30	D
E4-SS	Valparaiso	Valparaiso, Chile	03/03/1985	Llayllay	260.80	D
E5-SS	Chi-Chi	Chi-Chi, Taiwan	21/09/1999	Taichung	272.97	D
E6-SS	Ionian	Ionian, Greece	04/11/1973	Lefkada-OTE Building	207.00	D
E7-SS	Bucharest	Vrancea, Romania	04/03/1977	Bucharest-Building Research Institute	130.00	E
E8-SS	Tohoku	Tohoku, Japan	11/03/2011	Furukawa	208.05	D

[a] Velocity of propagation of shear waves measured up to 30 meters deep from the surface. Data obtained from NEHRP [18].
[b] Classification of soil type: soft soil (E), stiff soil(D). Data obtained from Table 7.

The ground motions considered in the study were classified according to the propagation rate of shear waves V_{S30} (measured up to 30 m deep from the surface), following the criteria set out in the National Earthquake Hazards Reduction Program [18], which have been been adopted by building codes such as the International Building Code [2].

Table 7
Site classifications using V_{S30} as an indicator of site response [18].

Soil type	General description	V_{S30} in m/s
A	Hard rock	> 1500
B	Rock	760 – 1500
C	Very dense soil and / or soft rock	360 – 760
D	Stiff soil	180 – 360
E	Soft soil	< 180
F	Special soils requiring site specific evaluations	

3.3 Intensity of ground motions

Based on the exposed in in section 6.2.1 of the NTC-DS of the RC-CDMX [1], the intensity of each seismic excitation was defined with a intensity spectrum of pseudo-accelerations whose ordinates are stablished by the following equation that incorporates both spectral components into a single intensity spectrum:

$$a_{es}(T) = \sqrt{a_{C1}^{2}(T) + a_{C2}^{2}(T)} \qquad (1)$$

In this equation a_{es} (T) is the ordinate of the spectrum that characterizes the intensity of seismic excitation; a_{C1} (T) and a_{C2} (T) are the spectral ordinates corresponding to the pseudo-acceleration elastic spectra, obtained for both horizontal components $C1$ and $C2$ employing a damping equal to 0.05 times the critical damping . Both spectral ordinates area evaluated at the vibration period T.

3.4 Scaling of ground motions

For a given ground motion, the scale factor was selected such that the spectral ordinates computed with equation 1 resulted not less than 1.3 times the spectral ordinate of the elastic design spectrum for periods between 0.2 and 1.3 times the fundamental period of the structure. The intensity of each seismic excitation was applying a rotation (respect to the vertical axis) of the horizontal components to obtain the maximum ordinates with equation 1. The last criterion was used to define the directions of application of each of the ground motion used in the dynamic nonlinear analysis.

Table 8
Scale factors and other parameters, ground motions applied to the E4 model.

Id	Scale factor	$a_{max\ C1}$ [a] in gals	$a_{max\ C2}$ [a] in gals	$I_{A\ C1}$ [b] in cm/s	$I_{A\ C2}$ [b] in cm/s
E1-SH	4.7316e+00	0.1601	0.1456	8.71e+04	7.44e+04
E2-SH	1.3526e+03	0.1804	0.1375	5.35e+04	3.68e+04
E3-SH	6.0548e+03	0.1789	0.1210	5.87e+04	3.40e+04
E4-SH	1.2596e+02	0.1402	0.1390	6.53e+04	4.92e+04
E5-SH	1.0214e+02	0.1909	0.1717	1.38e+05	1.45e+05
E6-SH	1.4260e+02	0.1575	0.1458	1.45e+05	1.35e+05
E7-SH	1.2478e+02	0.2141	0.1875	1.60e+05	1,40e+05
E8-SH	1.4367e+02	0.1644	0.1126	6.92e+04	4.58e+04

[a] Maximum acceleration in the seismic record.
[b] Arias intensity normalized respect to π/2g factor.

Table 9
Scale factors and other parameters, ground motions applied to the E8 model.

Id	Scale factor	$a_{max\ C1}$ [a] in gals	$a_{max\ C2}$ [a] in gals	$I_{A\ C1}$ [b] in cm/s	$I_{A\ C2}$ [b] in cm/s
E1-SS	3.2061e-01	0.1531	0.1765	1.96e+04	2.06e+04
E2-SS	1.0702e+00	0.1939	0.3107	7.15e+04	8.33e+04
E3-SS	6.5486e-01	0.1438	0.2348	1.76e+04	5.84e+04
E4-SS	7.5869e-01	0.2665	0.3968	1.72e+05	2.13e+05
E5-SS	1.2201e+00	0.1615	0.2311	1.03e+05	1.48e+05
E6-SS	7.6139e-01	0.2761	0.3787	2,55e+04	4.18e+04
E7-SS	7.8401e-01	0.1632	0.1495	1.87e+04	2.79e+04
E8-SS	4.3444e-01	0.1902	0.2397	1.38e+05	1.64e+05

[a] Maximum acceleration in the seismic record.
[b] Arias intensity normalized respect to π/2g factor.

Tables 8 and 9 list the scaling factors applied to the seismic excitations used in the calculation of the nonlinear response of models E4 and E8, respectively. Both orthogonal horizontal components that define the seismic excitation were scaled with a same factor, which maintains the ratio between both. Figure 3 shows the response spectra corresponding to the seismic excitations affected by the scaling factors listed in Tables 8 and 9. The corresponding design spectra are also included in the same figure.

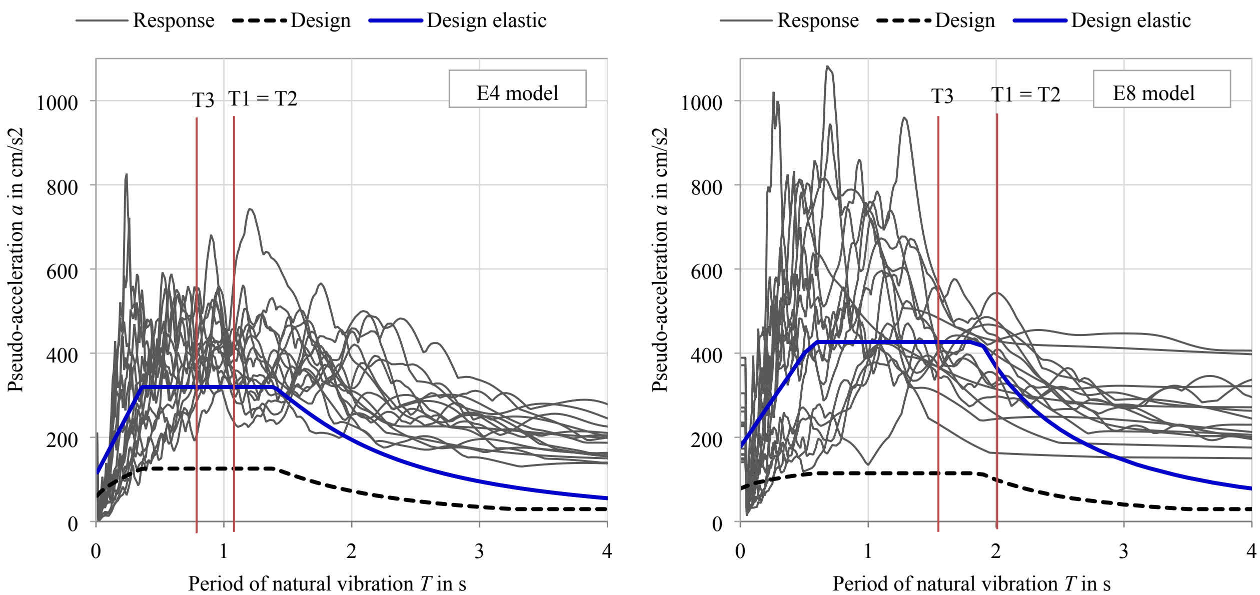


Figure 3. Spectra associated with seismic excitations used in the structural models ($\zeta = 0.05$).

4. Analysis

Each structural model was subjected to a specific set of seismic excitations as stated in section 6.2.1 of the NTC-DS of the RC-CDMX [1]. For each model and each excitation three-dimensional nonlinear dynamics analysis were performed, considering both orthogonal horizontal seismic components acting simultaneously. Each analysis was performed with the CANNY-E program [19], including second order effects. Damping fractions (proportional to mass and stiffness at each instant of time) equal to 0.05 times the critic, was used.

4.1 Structural system modeling

The columns were idealized with multi-spring models [20, 21] and the beams with uniaxial bending models. In all cases, post-yield stiffness equal to 10% of the initial stiffness was considered. The floor systems were modeled as rigid diaphragms and their rotation were included in the analysis.

The yield moments of fluence of the structural models were defined from the responses obtained from a spectral modal analysis, employing as seismic excitation the design spectrum. In the analysis the effect of gravitational loading was considered in addition to the effect due to lateral loading. With regard to bidirectional effects, these were considered without load factors. The effect of accidental torsion was estimated with accidental eccentricities on each interstory, by torsion moments applied at the level of slabs calculated with equation 2. As for the accidental eccentricity values e_{ai} three cases were considered for each analysis direction: 1) e_{ai} variable, from $0.05b$ to $0.1b$ (calculated according to equation 3), 2) e_{ai} equal to $0.1b_i$ and 3) e_{ai} equal to $0.05b_i$.

Thus, three different designs were obtained for each model. The dynamic analysis of the proposed structural models (E4 and E8) involves the analysis of each design proposal, considering the effect of accidental torsion in accordance with the procedures described in Sections 5 and 6.

5. Accidental torsion by accidental eccentricities

In seismic design, the resistance of the buildings is estimated to support accidental torsion resulting from mass variation and rigidity, mainly. In this sense, the RC-CDMX [1] estimates the resistance that must be provided to buildings with the help of accidental eccentricity e_a, which by multiplying it by interstory cuts leads to torsion moments of torsion M_{to}. With these latter can estimate torsion moments at the level of M_0 slabs. The regulation uses the load condition defined by floor moments M_0 of all levels as a load condition that is combined with the lateral load condition to calculate the forces and displacements that the building must withstand.

The RC-CDMX [1] considers two settings of torsion moments due to accidental eccentricity: one taking the floor moments with a positive sign and one with a negative sign. This condition is met according to the following equation:

$$M_{0i} = \pm \left[M_{ai} - M_{a(i+1)}\right] \quad (2)$$

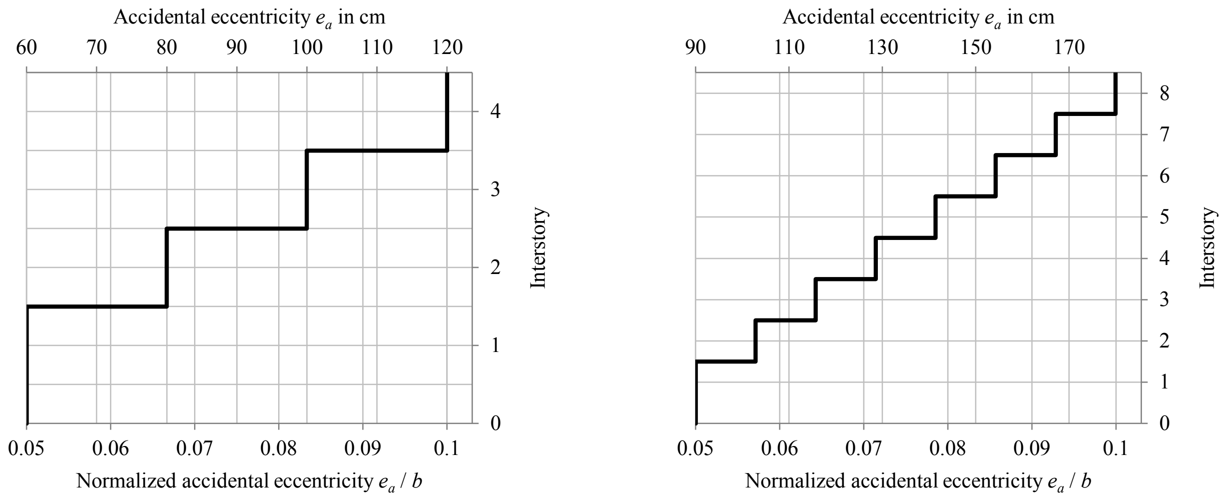
Where M_{0i} is the moment applied on the floor of the i-th level, and M_{ai} is equal to $V_i \cdot e_{ai}$, where V_i is the shear force of the i-th interstory in the direction of analysis, and e_{ai} the corresponding interstory accidental eccentricity.

The expression of the accidental eccentricity that recommends the RC-CDMX [1] takes into account the results of previous researches (*e. g.*, [22]) which emphasize that the accidental eccentricity is greater in the upper interstories of a building that in the lower ones. The following expression assigns an eccentricity equal to $0.05b$ to the lower interstory and $0.1b$ to the upper interstory, with a linear variation between:

$$e_{ai} = \left[0.05 + 0.05(i - 1)/(n - 1)\right] b_i \quad (3)$$

In equation 3, e_{ai} is the accidental eccentricity in the i-th interstory, n is the total number of floors of the structural system and b_i is the dimension of i-th interstory in the perpendicular direction to the direction of analysis.

Figure 4 shows interstory accidental eccentricities calculated with equation 3. Because both structural models have aspect ratios (width/length) equal to one at all levels, the interstory accidental eccentricity values are the same for both analysis directions.



a. E4 model.

b. E8 model.

Figure 4. Accidental eccentricity values e_a used in the study (calculated with equation 3).

In this study two approaches to include the effect of accidental torsion on the dynamic nonlinear analysis by accidental eccentricities e_a are used. The first one uses the load condition defined by equation 2 as an initial load condition whose response is calculated at the beginning of the dynamic analysis. In the definition of floor moments M_0, the interstory shear forces V are estimated within the step-by-step nonlinear analysis using as seismic excitation the horizontal components of the ground movement *C1* and *C2* individually (according to the analysis direction). This led to two independent sets of interstory shear forces, one for each direction of analysis. As a result of the condition defined by equation 2 and the simultaneous action of the two horizontal components of ground movement, four load conditions were available for each seismic excitation considered in the dynamic analysis of the proposed structural models.

In the second approach, accidental torsion is included by moving the position of the center of mass at the i-th floor a distance d_i (*e. g.*, [23]). This is obtained from the relationship between the floor moment M_{0i} and the lateral force F_i applied on the i-th level. In this case, the interstory shear forces V used in the calculation of floor moments M_0 and lateral forces F are estimated from a spectral modal analysis, using as seismic excitation the corresponding to the seismic action of design (design spectrum). Under these considerations, accidental torsion in the analysis model is determined by lateral force at every instant of time.

Both approaches were applied by considering three accidental eccentricity distributions e_a: 1) variable e_{ai} from $0.05b$ to $0.1b$ (calculated according to equation 3), 2) e_a equal to $0.1b_i$ and 3) e_a equal to $0.05b_i$. The first case corresponds to interstory accidental eccentricities calculated as stated in the RC-CDMX [1]. The last two cases correspond to typical values recommended by some building codes, such as the Uniform Building Code [3] and the National Building Code of Canada [4].

6. Accidental torsion by simulation techniques

6.1. Monte Carlo simulation

In this study, the Monte Carlo method is used to simulate the accidental seismic torsion phenomenon and to include its effect in the dynamic nonlinear analysis of the structural models. Its application begins with the generation of random samples associated with each of the random variables considered in the study and defined with their corresponding probability distribution functions, *fdp*. The results obtained from the simulation process are used to compare them with current proposals set out in some building codes that use accidental eccentricities.

6.2. Random variables

In this work, the stiffness of the structural elements (beams and columns), the magnitude of the dead load as well as the magnitude and position of the instantaneous live load are considered as random variables.

According to studies carried out by De la Llera and Chopra [24], the stiffness of a steel structural element (assumed as the product between Young's module of material E and the moment of inertia of the cross section of the element $I_{nominal}$) presents a behavior of distribution normal with coefficient of variation equal to 0.08 and mean equal to the $E \cdot I_{nominal}$ product. As for the magnitude of the dead load, studies were considered on the development of load criteria (based on probabilistic concepts) for the ANS A58 [25] *standard,* which indicate that it has normal distribution with coefficient of variation equal to 0.1. The mean values were considered equal to 450 kg/m^2 and 400 kg/m^2 for all levels of models E4 and E8, respectively.

Based on the studies carried out by Ruiz and Soriano [26], the magnitude of the live load presents a gamma distribution behavior with coefficient of variation equal to 0.292. The mean value was considered equal to 120 kg/m^2 at all levels of both structural models, except for the roof level at which a value equal to 96 kg/m^2 was taken. These values correspond to the reduced live load defined according to the recommended of the ASCE/SEI 7 [16] *standard.* As for its position, this has normal distribution behavior with coefficient of variation equal to 0.074. The mean value for each analysis direction was taken equal to $0.5b_i$.

Table 10
Random variables.

Random variable	Fdp	Mean	Coefficient of variation
Rigidity of beams and columns [23]	Normal	$E \cdot I_{nominal}$	0.080
Magnitude of the dead load [24]	Gamma	400 kg/m^2 and 450 kg/m^2	0.100
Magnitude of the instantaneous live load [25]	Normal	120 kg/m^2 and 96 kg/m^2	0.292
Position of the instantaneous live load [25]	Normal	$0.05b_i$	0.074

Table 10 summarizes the list of random variables considered in the simulation process as well as the statistical parameters used in the generation of the corresponding random samples. In this study the number of simulations N was considered equal to 10,000. This is due to the similarity with some related studies such as those reported in [27, 28] where the Monte Carlo method is used.

7. Combination with other actions

Both structural models were analyzed considering the effect of gravitational loads (dead load and instant live load), in addition to the effects of seismic excitations described in Tables 5 and 6. Note that the generation of random samples for the case of gravitational load (with the data listed in the Table 10) allows to define both the magnitude and the position of its resultant in each level, but not its distribution. Accordingly, in the application of the Monte Carlo method is assumed a variation in plan at each level of both the dead load and the instantaneous live load so that the statistical parameters associated with its magnitude and position were similar to those assumed in the study.

8. Results

In this study, two procedures were employed to consider the effect of accidental torsion on the dynamic nonlinear analysis of the structural models. The first involves the use of accidental eccentricities; the second corresponds to the application of the Monte Carlo method, which involves the variation of parameters associated to masses and rigidities, which directly incise in the accidental torsion. The response of the structural models was evaluated for different resistance capacities, defined in function of different levels of accidental torsion. For each model, values of ductility demand and interstory distortions were obtained. Only results associated with ductility demands of beams are presented. Since the principle of strong-column weak-beam was considered in the design, columns practically behaved with linear behavior.

Figures 6 and 7 show the overall response variations are shown (in terms of beam ductility demands) corresponding to each of the design proposals of the E4 and E8 models, respectively. The results correspond to the case where the effect of accidental torsion was estimated with Monte Carlo simulations. The values are maximum values $\mu_{\phi max}$ of the arithmetic means of the responses obtained for each seismic excitation μ_{ϕ}. To exemplify the computation of these values, Figure 5 shows the distributions of ductility demands obtained from the results of the Monte Carlo simulations. In general, multimodal distributions are obtained because the results correspond to responses from various seismic excitations. This is illustrated in Figure 5a for the case of a beam (arbitrarily selected). The ductility demand distributions shown in the other histograms of Figure 5 were obtained by analyzing separately each seismic excitation. Each one identifies the mean value μ_{ϕ}. The maximum of these values (defined as $\mu_{\phi max}$) corresponds to the indicated point and Figure 7a. This Figure 5 illustrate the computation procedure used to obtain the results shown in Figures 6 and 7.

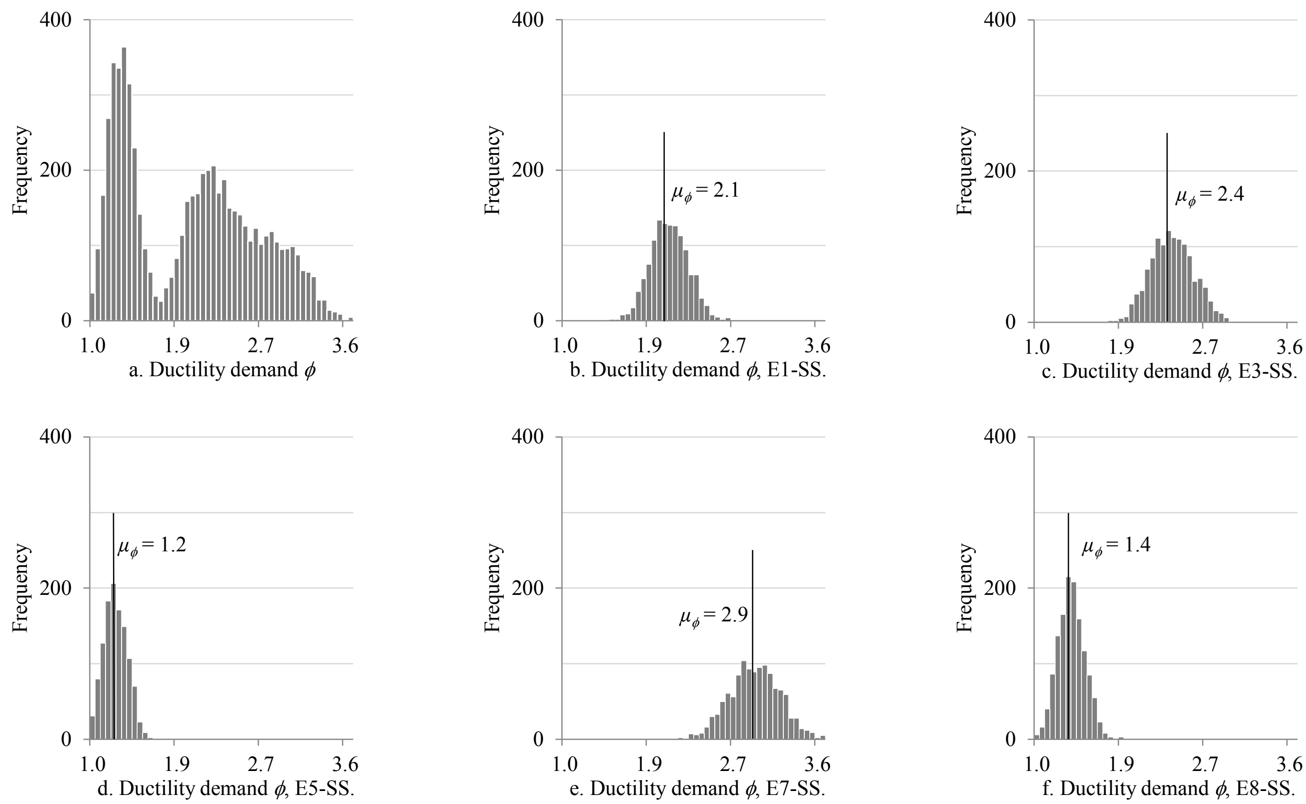


Figure 5. Distributions associated with beam ductility demands of E8 model.

In order to evaluate each of the design proposals the following figures also include the responses of the *reference* models, which do not include accidental torsion. The values presented correspond to maximum values of ductility demands, taken with respect to the ductility demands calculated for each seismic excitation. In this study, it is assumed that a design accidental-eccentricity value is appropriate when leads to ductility demands similar to those obtained for the *reference* model. This suggest that the design accidental eccentricity used in the design process provides the necessary strength to compensate for the *simulated* effects of accidental torsion.

Notice in these figures (6 and 7) that the ductility demands at the ridges of the curves correspond to the edge beams, while the values at the valleys correspond to the inner beams. In principle, this variation is due to the resistance and stiffness provided by the different frames that make up the structural system of the E4 and E8 models, are different. This leads to variations such as those corresponding to reference models in which the effect of accidental torsion is not considered. For cases that consider the effect of accidental torsion a similar variation is observed, however, the ridges and valleys tend to accentuate or reduce because of accidental torsion.

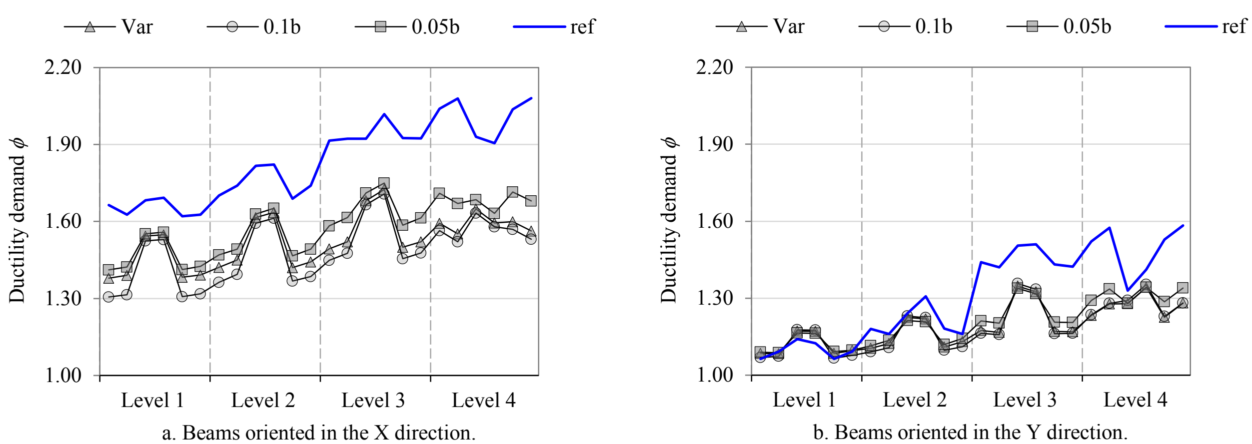
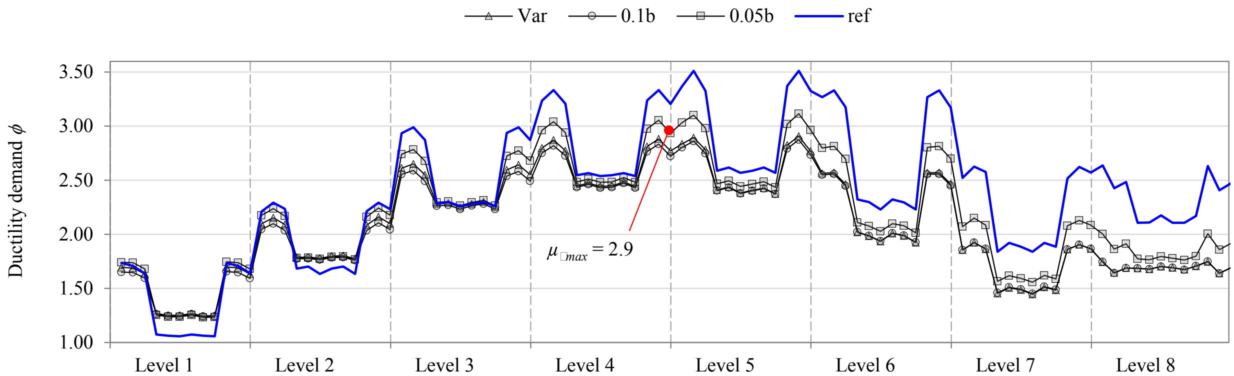
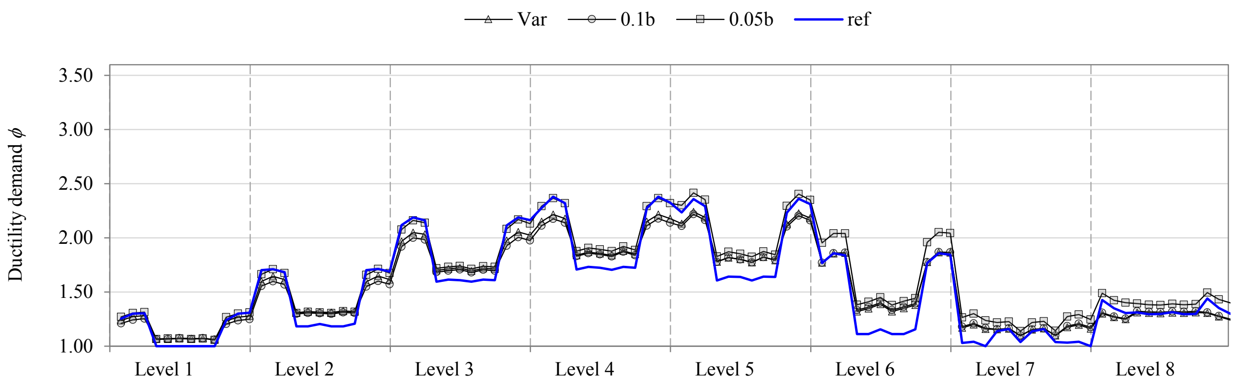


Figure 6. Global variation of ductility demands on beams for the different design proposals associated to E4 model.



a. Beams oriented in the X direction.



b. Beams oriented in the Y direction.

Figure 7. Global variation of ductility demands on beams for the different design proposals associated to E8 model.

From the results shown in Figures 6 and 7, it is observed that employing an accidental eccentricity e_a equal to 0.05b leads to designs that present higher ductility demands. An accidental eccentricity equal to 0.1b implies a resistant design. In general, similar values of ductility demands are obtained. In general, the ductility demands corresponding to design proposals associated to E4 model, they are no more than 3%; as for the E8 model there are differences of up to 6%. While the ductility demands of structural models *designed* with different accidental eccentricity values do not differ significantly, the responses of models *designed* with an accidental eccentricity e_a equal to 0.05b have less variability than those obtained from the *reference* models. Considering the latter case, for the E4 model, the difference in the response expressed as a percentage of the response obtained from the *reference* model results in 14% in the X

direction and 8% in the Y direction. For the E8 model, there are differences of 10% in the X direction and 8% in the Y direction.

A second procedure considers the effect of accidental torsion (in the analysis model) from accidental eccentricities. In the Figure 8 shows overall response variations (measured in terms of ductility demands by beams) for each of the design proposals associated with the E4 model. The results correspond to the case that considers accidental torsion using initial load conditions, *equivalent* to the effect of accidental torsion. In the Figures 9 and 10 show overall response variations for each of the design proposals associated with the E4 and E8 models, respectively. The results shown, correspond to the case that considers accidental torsion by moving the position of the center of mass at each level respect to torsion center. The values correspond to maximum values of ductility demands ϕ_{max}, taken respect to the values of ductility demands ϕ calculated to each excitation. In all cases, in addition to the results of the *reference* models and the Monte Carlo simulations, the results corresponding to the sum of the mean value $\mu_{\phi max}$ included, and the value of the standard deviation $\sigma_{\phi max}$ associated with the set of responses used in the definition of the value $\mu_{\phi max}$.

From the results shown in the Figures 8 and 9 it is observed that the way in which the effect of accidental torsion is considered in the analysis model leads to significant differences in the response of the structural model. In particular, for the E4 model, the use of an initial load condition *equivalent* to the effect of accidental torsion implies an increase in ductility demands of up to 65% respect to those obtained by moving the position of the center of mass in each level (respect to the torsion center). In global terms, are present increases in the response of approximately 45%. Such differences occur regardless of the level of accidental torsion considered in the analysis.

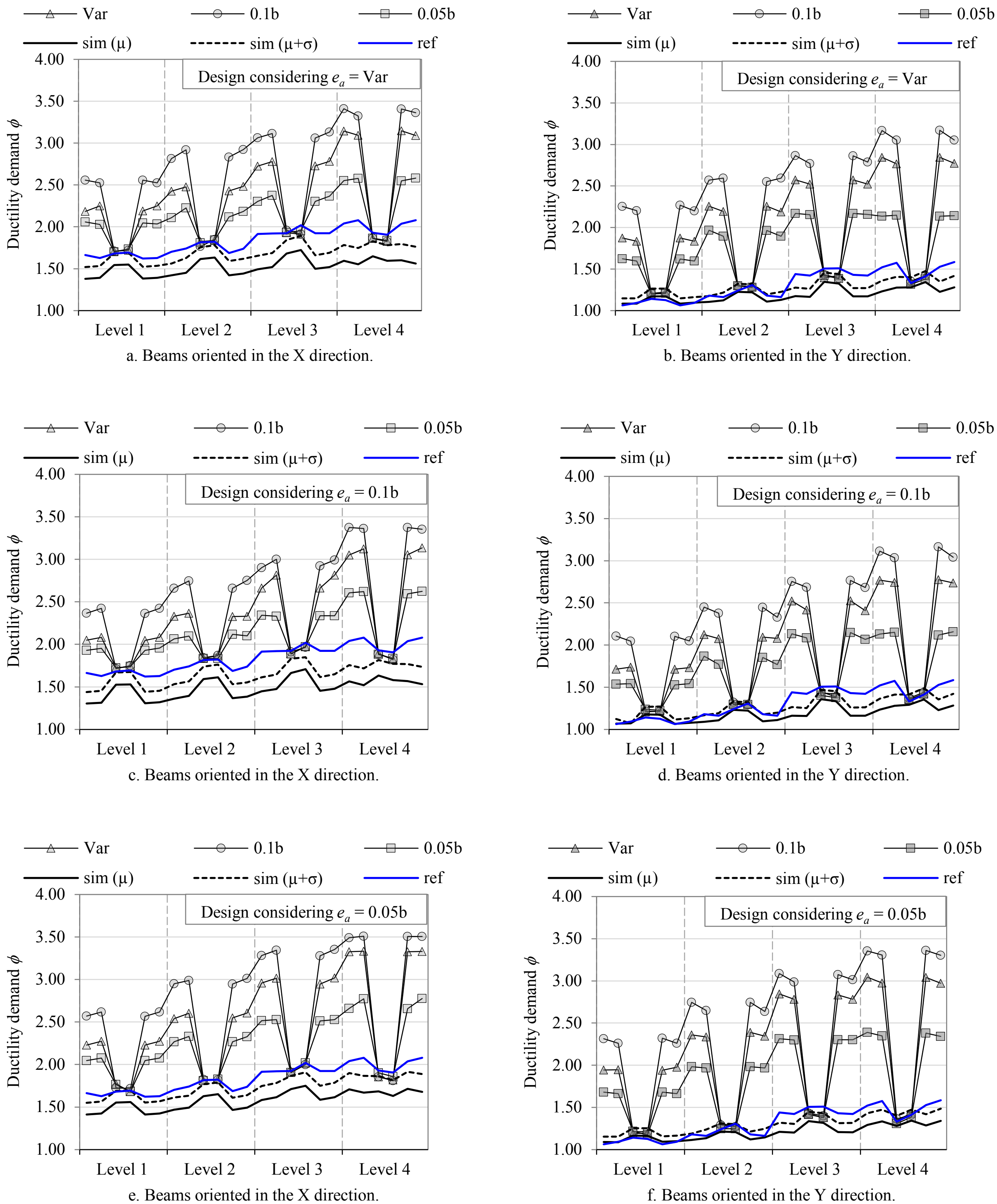
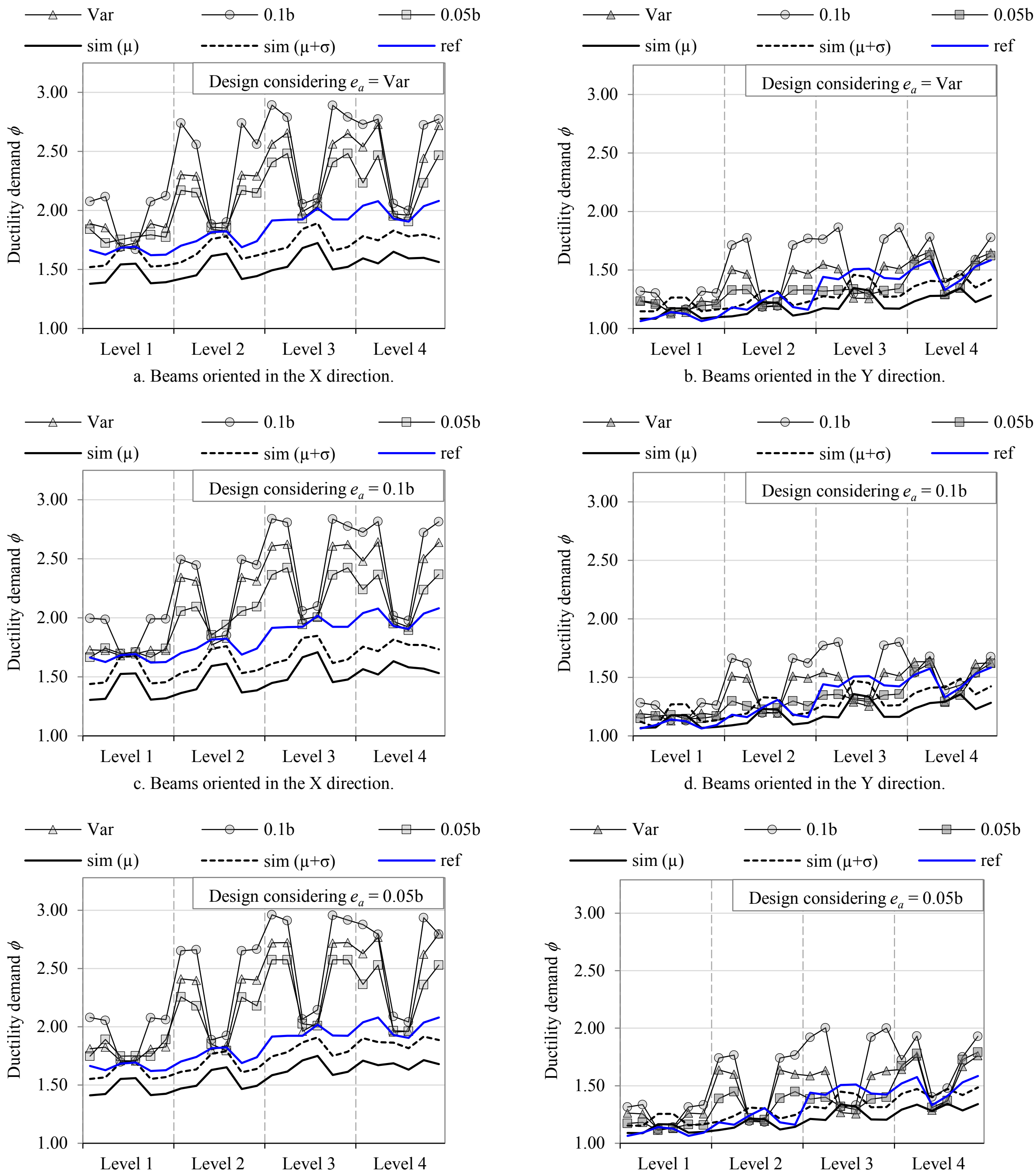


Figure 8. Global variation of ductility demands on beams for different levels of accidental torsion (estimated by initial load conditions), E4 model.



a. Beams oriented in the X direction.
b. Beams oriented in the Y direction.
c. Beams oriented in the X direction.
d. Beams oriented in the Y direction.
e. Beams oriented in the X direction.
f. Beams oriented in the Y direction.

Figure 9. Global variation of ductility demands on beams for different levels of accidental torsion (estimated displacing the position of the center of mass), E4 model.

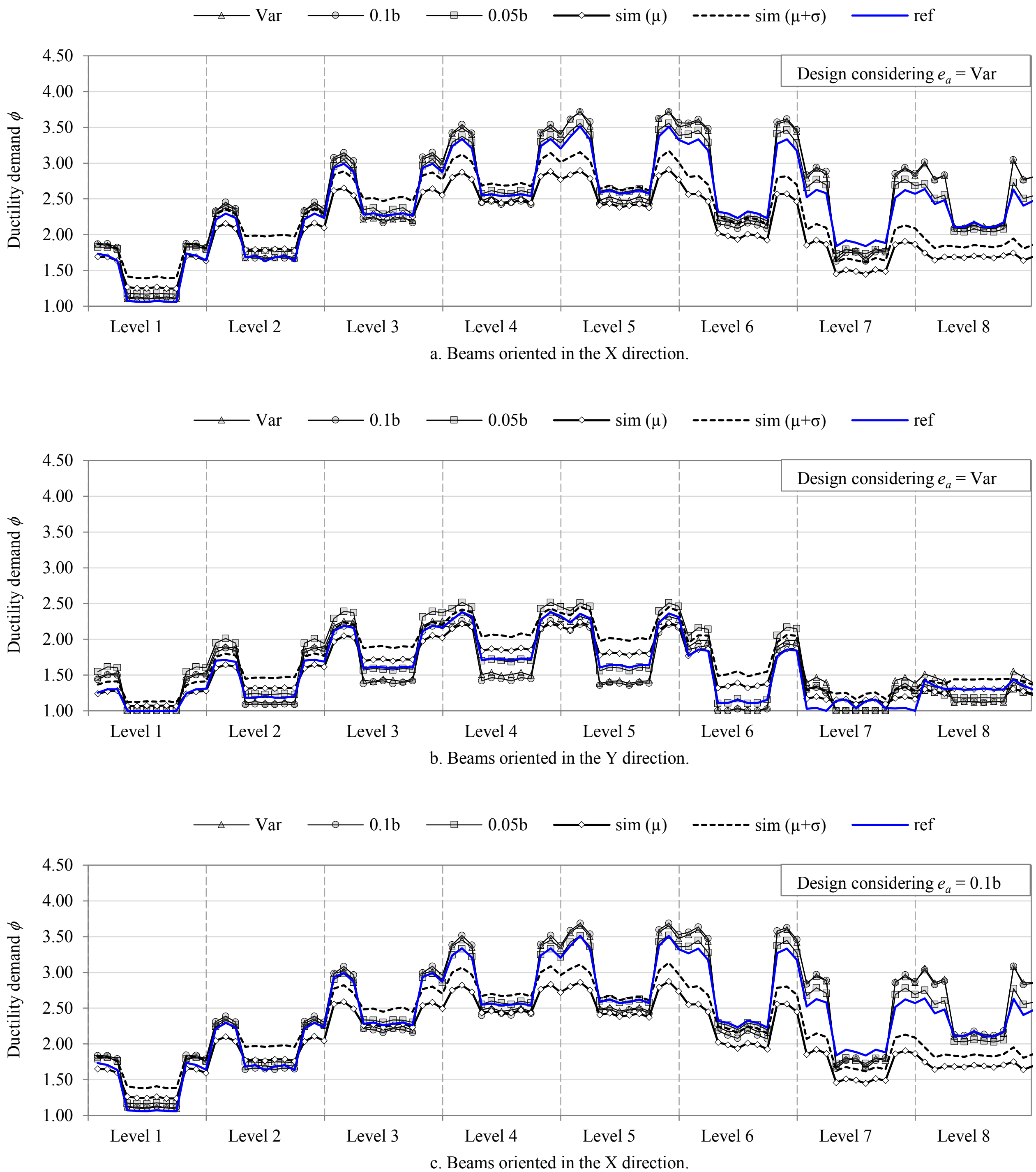
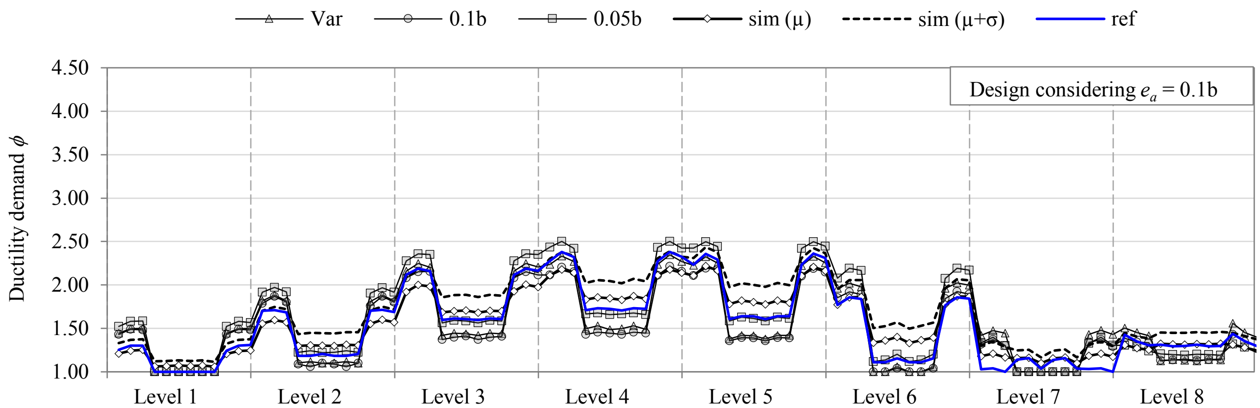
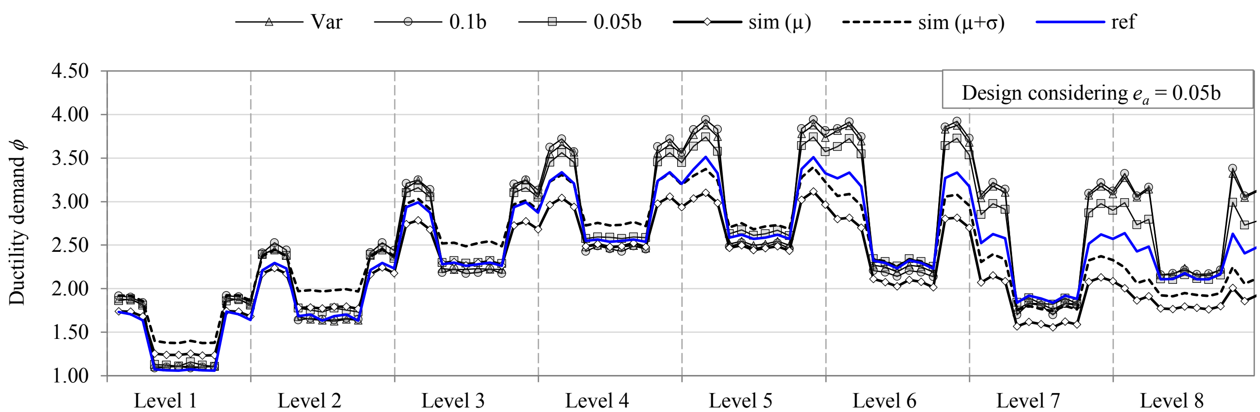


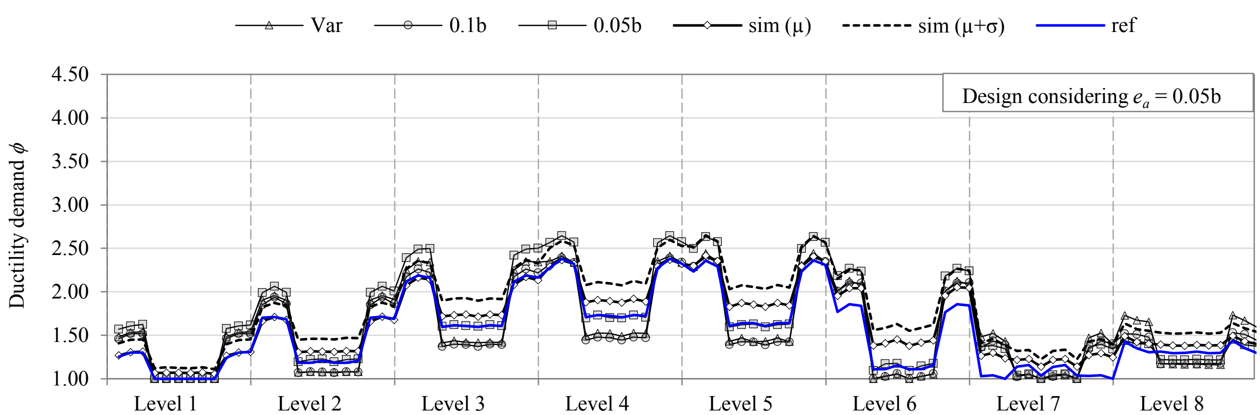
Figure 10-1. Global variation of ductility demands on beams for different levels of accidental torsion (estimated displacing the position of the center of mass), E8 model.



d. Beams oriented in the Y direction.



e. Beams oriented in the X direction.



f. Beams oriented in the Y direction.

Figure 10-2. Global variation of ductility demands on beams for different levels of accidental torsion (estimated displacing the position of the center of mass), E8 model.

In the Figures 10-1 and 10-2 it is observed that regardless of the accidental torsion design, it seems appropriate to consider an accidental eccentricity e_a equal to $0.05b$ in the analysis model to estimate the effect of accidental seismic torsion. A similar situation is observed in the Figure 9. The above is derived from the similarity between the ductility demands obtained in the structural models that consider an accidental eccentricity equal to 0.05*b* and those corresponding to the reference models. If the answers are compared to those obtained from the Monte Carlo simulations ($\mu_{\phi max} + \sigma_{\phi max}$), it is observed that the ductility demands for the different levels of accidental torsion considered in the study are superiors in the edge beams. The above suggests that it would be enough for these elements to use an accidental eccentricity e_a equal to 0.05*b*. However, the latter observation does not apply to interior beams in which the response is influenced by asymmetries in the rigidity resulting from Monte Carlo simulations. Such involvement is greater in the lower interstory than in the upper.

In terms of inter story lateral distortions, no significant differences were present. In general, these were virtually the same, regardless of the accidental eccentricity value used in the design of the structural models and the accidental torsion level considered in the analysis model. In global terms, differences of approximately 2% were presented. Comparing the responses with those obtained from the Monte Carlo simulations, differences of up to 5% were obtained in some interstory of E4 model and 7% in some interstory of E8 model.

In the Figure 11, is shown overall response variations (measures in terms of interstory distortions) of model E8. Only the following are shown in the design cases for which the lateral distortions of the interstory were greater. These are those for models *designed* with an accidental eccentricity equal to 0.05*b*. The values presented correspond to the average of the maximum distortions obtained on each interstory according to different levels of accidental torsion.

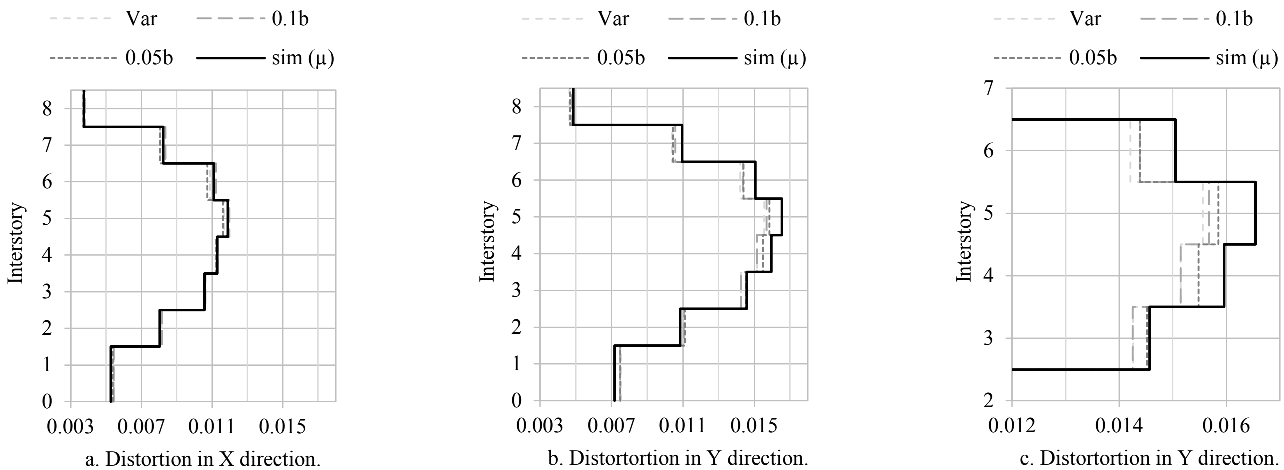


Figure 11. Lateral distortions, E8 model.

9. Conclusions

In this work a comparative analysis was carried out between the responses obtained with two procedures to consider the effect of accidental torsion on the dynamic nonlinear analysis: 1) applying the criteria established in typical building codes that use design accidental eccentricities and, 2) using simulation techniques.

Ductility demands for building models designed with three different distributions of (design) accidental eccentricities are practically equal each other, when Monte Carlo simulation is used. This suggest that the selection of a particular value of accidental eccentricity (between the range of $0.05b$ and $0.1b$) does not have a significant effect on the structural response. However, the use of an accidental eccentricity $e_a = 0.05b$ leads to ductility demands nearest the obtained from *reference* models, in which the effect of accidental torsion is not considered.

Differences on ductility demands of up to 65% were observed between both approaches used to include accidental torsion with accidental eccentricities [the first with torsional moments applied at the slabs as indicated by the equation (2), and the second with eccentric masses at each floor].

The results corresponding to the approach that uses torsional moments applied at the slabs are not consistent with the results obtained from Monte Carlo simulations. On the other hand, when the accidental torsion is included with eccentric masses at each floor the ductility demands are similar to those computed with the Monte Carlo simulations.

For analysis purposes, the use of mass eccentricities equal to $0.05b$ leads to ductility demands that are almost equal (but larger than) the ductility demands computed with the Monte Carlo method. When values of eccentricities larger than $0.05b$ are used in the analysis, the computed demands result even larger than those obtained with the Monte Carlo method. This suggests that in similar studies of accidental torsion, instead of using simulation techniques it is recommended to use eccentric masses with $e_a = 0.05b$.

It should be noted that the scaling procedure of earthquake records suggested by the RC-CDMX [1] is part of a revision procedure of some buildings. A similar scaling procedure is included in the ASCE/SEI 7 [26] *standard*.

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ACCIDENTAL TORSION WITHIN THE FRAME OF NONLINEAR DYNAMIC ANALYSIS USING CODE ACCIDENTAL ECCENTRICITIES AND MONTE CARLO SIMULATIONS

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Declaration of interests

☒ The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

☐ The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: