UNIVERSIDAD AUTÓNOMA DEL ESTADO DE MÉXICO

Facultad de Ingeniería

EVALUACIÓN DE LA RESPUESTA INELÁSTICA DE MODELOS ESTRUCTURALES DE EDIFICIOS EN FUNCIÓN DE LA COMBINACIÓN DE EFECTOS SÍSMICOS ORTOGONALES.

TESIS

Que para obtener el grado de

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Presenta

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DEDICATORIA

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RESUMEN

Se estudió el comportamiento inelástico de varios modelos estructurales con diferentes períodos de traslación en ambas direcciones ortogonales. Para cada modelo, el ángulo de incidencia del sismo varió de 0° a 90° con incrementos de 10°. La combinación de los efectos sísmicos ortogonales horizontales utilizados para el diseño varió para α=10%, 30%, 50%, 70% y 100%. Se analizaron doce registros sísmicos, 6 de suelo firme y 6 de suelo blando. Se utilizaron dos factores de reducción de las fuerzas elásticas de diseño (Q=2 y 4). En base a estos parámetros, se realizaron diferentes análisis. Para cada uno, se evaluaron las demandas de ductilidad de columnas y vigas, así como las distorsiones de entrepiso. Con estas distorsiones de entrepiso, se estimó el daño de los modelos para cada caso. Se evaluó el costo total de los modelos, considerando el costo inicial del edificio, los costos de reparación, las pérdidas de contenido, los costos de pérdida por rentas y las pérdidas debidas a personas heridas y fallecidas. El factor principal que afecta el costo total de los modelos fue el tipo de suelo, sin embargo, las otras variables también modifican la estimación del costo total. Para suelos firmes, el promedio del α óptimo es 0.85 para Q=2 y 0.9 para Q=4. Para suelos blandos el promedio α óptimo es 0.20 para Q=2 y 0.30 para Q=4. Las demandas de ductilidad también varían fuertemente en función de las variables estudiadas La variable principal que afecta las demandas de ductilidad es el ángulo de incidencia. Los valores de α que se aproximan en mayor medida a las demandas de ductilidad calculada con los valores de Q fueron los siguientes: para suelo firme, 0.1 para Q=2 y 0.5 para Q=4; para suelo blando, 1.0 para Q=2 y 0.7 para Q=4.

ABSTRACT

The inelastic behavior of several structural models with different translation periods in both orthogonal directions was studied. For each model the incidence angle of the earthquake was varied from 0° to 90 ° with increments of 10°. The combination of the horizontal orthogonal seismic effects used for design varied for α = 10%, 30%, 50%, 70% and 100%. Twelve seismic records, 6 of firm soil and 6 of soft soil, were analyzed. Two reduction factors of design elastic forces were used (Q = 2, and 4). Based on these parameters, different analyses were carried out. For each one, the ductility demands of columns and beams were evaluated, as well as interstory drifts. With these interstory drifts, the damage of the models was estimated for each case. The total cost of the models was evaluated, considering the building initial cost, repair costs, content losses, income loss costs, and losses due to injured and deceased people. The principal factor that affects the total cost of the models was the type of soil, however the other variables also modify the estimation of the total cost. For firm soil, the average of the optimum α is 0.85 for Q=2 and 0.9 for Q=4. For soft soil the optimum α average is 0.20 for Q=2 and 0.30 for Q=4. The ductility demands also strongly vary as a function of the studied variables. The main variable that affects ductility demands is the incidence angle. The α values that approximate to a greater extent the computed ductility demands with the Q values were as follows: for firm soil, 0.1 for Q=2 and 0.5 for Q=4; for soft soil, 1.0 for Q=2 and 0.7 for Q=4.

PRESENTACIÓN

El movimiento sísmico del suelo posee seis componentes, tres de traslación y tres de rotación. Dada la manera en que se registran el movimiento del suelo, es común analizar las estructuras bajo la acción de los componentes ortogonales horizontales de traslación. Si la estructura está ubicada cerca del epicentro del sismo, debe incluirse la componente vertical del sismo.

El análisis convencional de las estructuras se realiza en forma individual para cada uno de los componentes del movimiento sísmico. Posteriormente, se combinan las máximas respuestas de cada componente, con el propósito de obtener la máxima respuesta al considerar los dos componentes del movimiento del suelo actuando de manera simultánea. Ante esta situación, la mayoría de las recomendaciones y códigos de diseño establecen reglas de combinación de los efectos sísmicos ortogonales. La regla de combinación más utilizada es la que establece el uso del 100% de la respuesta debida a la acción de una componente del movimiento del sismo más un porcentaje α de la componente ortogonal del movimiento sísmico.

Las reglas de combinación de efectos sísmicos ortogonales más utilizadas son las del 100% + 30% y 100% + 40%. Existen otras reglas de combinación tales como la SRSS, la cual estima la máxima respuesta bidireccional como la raíz cuadrada de la suma de las máximas respuestas unidireccionales elevadas al cuadrado; la regla CQC3, la cual toma en cuenta explícitamente la correlación entre las respuestas modales y las componentes horizontales del movimiento del suelo. Para el caso de la ciudad de México, las normas técnicas complementarias para diseño por sismo señalan la regla de combinación 100% + 30%.

Todas estas reglas de combinación fueron propuestas bajo las siguientes consideraciones: 1) las recomendaciones se obtuvieron a partir de estudios elásticos, 2) no consideran el tipo de terreno en el que se ubican las estructuras, 3) No consideran las propiedades dinámicas de las estructuras tales como el periodo lateral de traslación, etc. Diversos trabajos han estudiado el porcentaje de combinación de efectos sísmicos ortogonales α, sin embargo, no se han efectuado estudios que relacionen dicho porcentaje con los costos totales de las edificaciones. Por lo cual, este trabajo estudia el costo total de las edificaciones integrado por: costo inicial, costos de reparación de daños, perdidas en los contenidos del edificio, perdidas por rentas, perdidas por personas heridas y las perdidas por personas fallecidas.

Se presenta una comparación de la respuesta inelástica de distintos modelos estructurales. Las variables consideradas son: valores del porcentaje de combinación de efectos sísmicos ortogonales α, diferentes periodos de traslación de las estructuras, varios registros sísmicos, diversos ángulos de incidencia del sismo y dos tipos de terreno (firme y blando). El análisis permitió obtener demandas de ductilidad de trabes, columnas y distorsiones de entrepiso de los modelos estructurales. Mediante los niveles de distorsión de entrepiso de los modelos estructurales, se evaluó el costo total de las edificaciones.

El documento se organiza en tres capítulos. El primero contiene el protocolo de tesis aprobado. El segundo muestra la evidencia del envío del articulo a la revista seleccionada. Finalmente, el tercer capítulo contiene el artículo de investigación elaborado.

CONTENIDO

PARTE 1

PROTOCOLO DE TESIS

PROTOCOLO DE TESIS.

I.- Antecedentes generales

El movimiento sísmico del terreno en un sitio se define mediante la traslación y rotación a lo largo de los ejes ortogonales, x, y, z. Así mismo mediante el espectro de respuesta se estiman los valores máximos de las respuestas de los componentes individuales de la excitación sísmica, mismos que no suelen producirse al mismo tiempo, por lo que se busca combinarlos a fin de estimar la respuesta máxima al movimiento del terreno (Chopra, 2014).
Acción sísmica en las construcciones

A fin de dar un tratamiento aproximado de una excitación sísmica en las estructuras, se considera lo siguiente:

- El movimiento del terreno puede transformarse en un conjunto ortogonal de ejes denominados 1 y 2, cuyos componentes de aceleración no se consideran correlacionados.
- Se denominan a estos, ejes principales (mayor y menor) del movimiento del terreno.
- El componente principal mayor está dirigido aproximadamente al epicentro del sismo y es horizontal, mientras que el componente principal menor también es horizontal y perpendicular al mayor (Chopra, 2014).

La forma en que un sismo incide en una estructura está planteada mediante la figura 1.

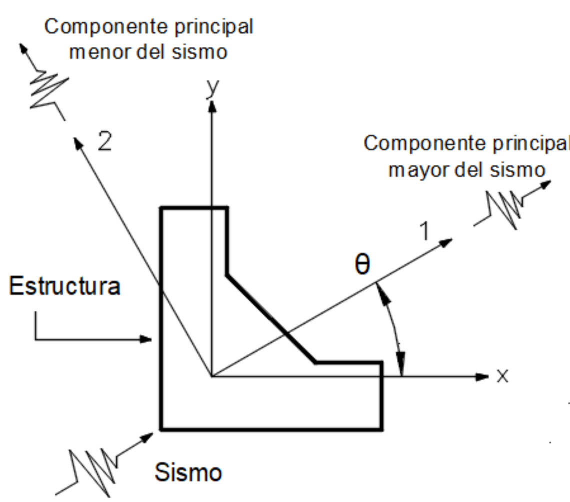

Figura 1. Relación de ejes de referencia.

Donde los ejes x-y corresponden a los ejes de la estructura, mientras que los ejes 1 y 2 son los correspondientes al movimiento del terreno. La relación entre los dos ejes de referencia se da por medio del ángulo θ, denominado ángulo de incidencia de las ondas sísmicas, ambos ejes de referencia están correlacionados (Chopra, 2014).

Reglas de combinación de efectos sísmicos ortogonales

Debido a que la dirección en que las ondas sísmicas inciden en una estructura es incierta, la máxima respuesta se obtiene mediante reglas de combinación de los efectos sísmicos ortogonales. Una forma muy empleada es la combinación de las respuestas unidireccionales de los sismos que inciden en la estructura. Dichas reglas son adoptadas por los códigos de diseño sísmico de diferentes asociaciones y países.

En la mayoría de los códigos de diseño sísmico se considera la regla de combinación del 100% de la máxima respuesta debido a la acción del sismo en una dirección (respuesta unidireccional), y α (parámetro en porcentaje) veces la máxima respuesta causada por la acción del sismo en la dirección ortogonal. Las reglas anteriores suponen que los ejes principales del movimiento del terreno coinciden con los ejes principales de la estructura y que ambos componentes del movimiento del suelo tienen la misma intensidad, tales reglas son conocidas como las de 100% + α.

Así mismo se tienen otros métodos para combinar las respuestas tales como la SRSS que estima la máxima respuesta bi-direccional como la raíz cuadrada de la suma de las máximas respuestas unidireccionales elevadas al cuadrado, además de la regla CQC3 que toma en consideración el ángulo de incidencia del movimiento sísmico (Chopra, 2014).
No obstante, estas reglas de combinación de efectos sísmicos ortogonales poseen limitación, ya que son empleadas considerando análisis elásticos de las estructuras, dejando de lado el comportamiento inelástico de las mismas, el cual es muy probable que se presente ante un evento sísmico de mayor magnitud (Rigato, 2007).

Análisis no lineal y el programa Canny-e.

La elasticidad es la propiedad que hace que un cuerpo que ha sido deformado regrese a su forma original después de ser removidas las fuerzas deformadoras (Fitzgerald, 1996). Así pues, en un sistema estructural se tiene un análisis lineal siempre que el desplazamiento sea una función lineal de la fuerza aplicada en la estructura, cuando esto no se cumple, el análisis se vuelve no lineal o inelástico (De la Colina, et al., 2016).

El análisis no lineal de una estructura permite determinar si las cargas actuantes provocan que los elementos excedan sus límites de comportamiento lineal o no, y en caso de hacerlo, debe ser capaz de describir su respuesta en este intervalo (Dionicio, 2011).

El programa CANNY-E, para análisis no lineal de estructuras tridimensionales, considera en su análisis el modelo de Resortes Múltiples (mRM) y es capaz de considerar la aplicación de acciones estáticas y dinámicas. Fue desarrollado para analizar, bajo el enfoque de macroelemento, estructuras de concreto reforzado, acero o compuestas, con o sin presencia de muros además de ser diseñado para trabajar con una baja demanda de memoria optimizando los procedimientos de cálculo.

II.- Problema de estudio

En el diseño de estructuras se emplean combinaciones de efectos sísmicos ortogonales para estimar su respuesta, sin embargo, no existe consenso acerca de la combinación que arroje mejores diseños en función de la demanda de ductilidad que desarrolle la estructura.

III.- Justificación

Actualmente no existe consenso en las investigaciones acerca de la mejor combinación de efectos sísmicos ortogonales, así como de sus implicaciones en el análisis, diseño, nivel de daño y costos de construcción de las estructuras.

Este trabajo pretende dar recomendaciones sobre el uso de las combinaciones de los efectos sísmicos ortogonales, enfatizando aquellas que garanticen un nivel de daño mínimo, de acuerdo con las características de los modelos estudiados.

IV.- Objetivo.

Evaluar la respuesta inelástica de un modelo estructural (con periodo variable en cada dirección) en función de la combinación de los efectos sísmicos ortogonales de diseño.

Objetivos particulares

1. Evaluar los desplazamientos laterales del modelo estudiado en función de la combinación de los efectos sísmicos ortogonales utilizada en su diseño.

2. Evaluar las demandas de ductilidad del modelo, con el propósito de estimar el nivel de daño que presenten en función de la combinación de los efectos sísmicos ortogonales utilizada en su diseño.

3. Emitir recomendaciones acerca de las reglas de combinación de los efectos sísmicos ortogonales, con el propósito de garantizar mínimos niveles de daño.

V.- Hipótesis

La regla de combinación de los efectos sísmicos ortogonales 100% +30% utilizada en el diseño de una estructura, subestima la respuesta inelástica de la misma en un 40% y no garantiza el menor daño estructural.

VI.- Revisión de bibliografía y Estado del Arte

A fin de estimar la máxima respuesta en una estructura, diversos investigadores han realizado propuestas acerca de cómo llevar a cabo la combinación de las respuestas unidireccionales y emplearlas en modelos estructurales. Rosenblueth y Contreras (1977) propusieron un valor de α=30%, por su parte, Newmark (1975) propuso un valor de α=40%. Adicionalmente, algunos códigos especifican una tercera regla, la cual estima la máxima respuesta bi-direccional como la raíz cuadrada de la suma de las máximas respuestas unidireccionales elevadas al cuadrado (SRSS).

Con base en el trabajo de Smeby y Der Kiureghian (1985), Menun y Der Kiureghian (1998) propusieron una regla de combinación modal para sistemas elásticos (regla CQC3), la cual toma en cuenta explícitamente la correlación entre las respuestas modales y la correlación entre los componentes horizontales del movimiento del suelo. La ecuación que proporciona esta regla para estimar la respuesta es función del ángulo de incidencia del temblor. López y Torrres (1997) desarrollaron una ecuación que permite obtener el ángulo de incidencia que produce la máxima respuesta al utilizar esta regla (Valdés, et al., 2015).

Derivado de las diferentes reglas de combinación de efectos sísmicos ortogonales diferentes países, organismos, asociaciones etc., han establecido en sus códigos de diseño la regla de combinación que consideran más conveniente. Se presenta en la tabla 1 las combinaciones de efectos sísmicos ortogonales que consideran algunos códigos de diseño.

Núm.	Código de diseño	Nombre y origen.	Regla de combinación
1	**NEHRP 1997**	National Earthquake Hazards Reduction Program. USA. Lineamientos para la rehabilitación sísmica de edificios. Código Estadounidense.	100% + 30%
2	**NEHRP 2003**	National Earthquake Hazards Reduction Program. USA. Disposiciones recomendadas para la regulación sísmica de nuevos edificios y otras estructuras	100%+30%
3	**COVENIN 2001**	Norma Venezolana COVENIN. Edificaciones Sismo resistentes.	SRSS, 100% +30% y la CQC3
4	**Caltrans. 2013**	California Department of Transportation Version 1.7. (USA-California). Seismic Design Criteria	100% +30%, CQC3-SRSS
5	**RCDF 2004**	México. Reglamento de Construcciones para el Distrito Federal. Normas Técnicas Complementarias para el diseño por Sismo.	100%+30%
6	**IBC 1997**	Uniform Building Code. USA.	100%+30%, opcionalmente SRSS
7	**ASCE/SEI 7-10 2010**	American Society of Civil Engineers Minimum. USA. Design Loads for Buildings and Other Structures	100%+30%
8	**ASCE/SEI 41-13 (2013)**	American Society of Civil Engineers. USA. Seismic Evaluation and Retrofit of Existing Buildings (Zimmerman, et al., 2014).	100%+30%
9	**FHWA-HRT-06-032 (2006)**	USA. Seismic Retrofitting Manual for Highway Structures. Part 1 – Bridges.	SRSS, 100%+40%
10	**AASHTO 2011**	American Association of State Highway and Transportation Officials, Washington, DC. USA. Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition (Zimmerman, et al., 2014).	100%+30%

Valdés y Ordaz, desarrollaron algunas expresiones analíticas para estimar la respuesta máxima combinada de estructuras elásticas, producida por las dos componentes horizontales de terremotos registrados en suelo blando. Para ello emplearon la teoría de vibraciones aleatorias, señalando la importancia del ángulo de incidencia del sismo y el tipo de respuesta considerado en términos de la dirección de sus componentes (ortogonales o co-lineales) (Valdés, et al., 2015).

K.G. Kostinakis, A.M. Athanatopoulou, V.S. Tsiggelis, estudian la eficacia de las reglas de combinación mediante un análisis de la historia de la respuesta lineal de estructuras; demostrando que las reglas del 30% y del 40% subestiman los valores máximos de la respuesta obtenida para todos los ángulos de incidencia en una estructura. Se observó que la respuesta de una estructura ante el uso de una regla de combinación está fuertemente afectada por el sistema de referencia que se seleccione (Kostinakis, et al., 2013).

Athanasios G. Tsourekas and Asimina M. Athanatopoulou, presentan la conveniencia del uso de las disposiciones de la Guía Regulatoria Nuclear respecto a la combinación de los efectos causados por las tres componentes de un sismo. Dichas disposiciones no producen resultados conservadores comparados con la máxima respuesta evaluada para cualquier ángulo de incidencia sísmica. Se encontró que la influencia de la excentricidad en las estructuras no influye de manera importante en la respuesta, además se encontró que el periodo natural de vibración de las estructuras y el tipo de sismo de estudio puede subestimar hasta un 40% la respuesta que establece la Guía Regulatoria Nuclear (Athanasios & Asimina, 2013).

Alfredo Reyes-Salazar, Federico Valenzuela-Beltrán, David de León-Escobedo, Eden Bojorquez-Mora y Arturo López Barraza, estudian la respuesta estructural de modelos de edificios de acero con marcos resistente a momento (MRF), considerando dos y tres componentes de los sismos. Se señala que la respuesta máxima no se presenta en la dirección principal de dichos componentes y su ángulo crítico varia de un sismo a otro, además la respuesta de la componente normal es mayor con respecto a la respuesta de la componente principal (principalmente en un comportamiento inelástico de las estructuras).

Así mismo estos autores señalan que las reglas de combinación (30% y SRRS) subestiman la carga axial y sobrestiman el cortante medio, dicha variación depende del grado de correlación de las componentes de los terremotos, del tipo de sistema estructural, de los parámetros de respuesta, de la localización del miembro estructural y del nivel de deformación de la estructura (Reyes, et al., 2016).

Leila Shahryari, Abdolrasoul Ranjbaran, Ali Mansoori realizan un análisis espectral lineal de varios modelos de edificios con y sin simetría geométrica y de rigidez lateral, mostrando que, para diferentes ángulos de incidencia de un sismo, se tiene un efecto mayor en la respuesta a carga axial de columnas comparados con elementos de contraviento y vigas de una estructura.

También se observó que la regla de combinación del 100%+30% sobrestima en mayor medida la respuesta en columnas de estructuras regulares, misma que se acentúa de manera proporcional al número de niveles de la estructura. Se proponen valores del parámetro de combinación α, en función de la forma geométrica de la planta de la estructura, el sistema resistente a carga lateral de la estructura, la localización de columnas en ejes arriostrados etc. (Shahryari, et al., 2013).

Julio J. Hernández, Oscar A. López, comparan las respuestas de las diferentes combinaciones de las componentes ortogonales de los sismos, utilizando las reglas de combinación RCSC, del 30%, 40% y una regla del UBC (IBC), con respecto a la respuesta crítica dada por la regla CQC3, considerando el factor de intensidad espectral $\gamma 2$ (relación entre los espectros de pseudoaceleraciones) entre las componentes horizontales del movimiento del suelo.

Se encuentra que para valores de la intensidad espectral $\gamma 2$ entre 0.5 y 0.8, las respuestas estimadas por la regla de RCSC y del 40% son conservadoras, la regla del 30% es intermedia y la regla IBC#1 subestima la respuesta para valores grandes de la respuesta vertical de un sismo (Hernández & López, 2002).

S.C Potnis, R.S. Desai y I.D. Gupta, proponen un método a fin de obtener la respuesta crítica ante la acción simultánea de dos componentes sísmicas mediante la superposición de espectros de respuesta resultantes de la aplicación de los registros sísmicos a la estructura en un rango de 0 a 180°. Así mismo se observó discrepancia en estudios anteriores que señalan a la respuesta crítica referida a los ejes principales de la estructura, mientras que otros señalan orientaciones diferentes. El método propuesto demuestra poseer niveles de sobreestimación bajos y aceptables (Potnis, et al., 2012).

Mahmood Hosseini y Ali Salemi llevan a cabo un análisis no lineal en la historia del tiempo de dos edificios con marcos resistentes a momento, usando las aceleraciones de las dos componentes del movimiento del suelo debidas a un sismo, variando el ángulo de incidencia de dichos sismos.

Observaron que las fuerzas internas de los elementos dependen del ángulo de incidencia del sismo con respecto a los ejes de la estructura, tomando como ejemplo la fuerza axial en columnas que arroja la respuesta más sensible al cambio de ángulo. El momento flexionante máximo en columnas ocurre principalmente para ángulos de 0 y 90 grados, así mismo dicho ángulo que produce la respuesta critica en una estructura, varia de un sismo a otro por lo que no hay un valor fijo que estime las respuestas máximas (Hosseini & A., 2008).

Ernesto Heredia Zavoni and Raquel Machicao Barrionuevo, estudian los efectos que producen las componentes horizontales del suelo, para sistemas estructurales lineales torsionalmente rígidos y flexibles, en terreno firme y blando para estructuras de un solo nivel, con ejes asimétricos.

Observando que el efecto de las componentes horizontales del suelo, para estimar las máximas respuestas en la estructura, es diferente de acuerdo con el sistema estructural (torsionalmente flexible o rígida), al periodo natural de traslación del sistema, a las condiciones del suelo (terreno duro o blando). Señalan que las reglas de combinación (30% y 40%) pueden ser muy conservadoras o subestimar la respuesta dinámica ya que no consideran las propiedades de la estructura y las condiciones del suelo en el que se ubica (Heredia & Machiao, 2004).

Oscar A. López, Anil K. Chopra and Julio J. Hernández, evalúan la exactitud de las reglas de combinación de efectos sísmicos ortogonales obtenidos mediante las reglas SRSS, 30%, 40% y SRSS simplificada, con respecto al valor dado por la regla CQC3 misma que considera la dirección y ángulo principal del movimiento del suelo que afecta a la estructura.

Las respuestas evaluadas por los autores fueron la fuerza axial y cortante en las direcciones X-Y de columnas, determinando que las regla: simplificada del SRSS, 30% y 40%sobreestiman la respuesta critica, mientras que la regla del SRSS subestima la respuesta crítica. Así mismo se señala que la respuesta crítica incrementa cuando los periodos de vibración de los dos modos que contribuyen a la respuesta de las componentes X-Y del movimiento del suelo son cercanos uno del otro. Se recomienda emplear la regla del CQC3 a fin de evitar los errores generados por las subestimaciones o sobreestimaciones de las reglas que se compararon (López, et al., 2001).
Ch.Ch. Mitropoulou y N.D. Lagaros realizaron bases de optimización para el diseño de miembros de acero y compuestos (acero y concreto), con referencia a los costos iniciales de construcción y al ángulo de incidencia sísmico, el cual es tratado mediante el método MIDA (Análisis Dinámico de Incremento de Multicomponentes).

Se planteó una función a minimizar (Costo Inicial) que considera la sección transversal de los elementos, costo de materiales, costos de labores de construcción y costos de elementos no estructurales. Se observó que la máxima respuesta sísmica está dada para diferentes ángulos de incidencia. En general el diseño de las columnas y vigas compuestas poseen un rendimiento mejorado, en comparativa con el diseño de marcos de acero, en cuanto al costo inicial no se presenta variación importante en la mayoría de los casos (Mitropoulou & Lagaros, 2016). Jinsuo Nie, Richard J. Morante, Manuel Miranda and Joseph Braverman, describen la manera adecuada de considerar la regla del 100%+40%+40%, toda vez que dicha regla de combinación es presentada de manera diferente en los códigos ASCE 4-98 y Regulatory Guide 1.92 (RG 1.92).

Una primera consideración señala que los valores absolutos de las respuestas de la RG 1.92 representan los dos casos extremos de las respuestas que establece la ASCE 4-98, mientras que una segunda consideración señala que para combinaciones intermedias dichas respuestas no gobiernan el diseño de la estructura. Se concluye que los dos formatos de presentación de la regla de 100%+40% son equivalentes siempre que esta se aplique a un solo parámetro de respuesta a la vez. Se menciona que la regla del 100%+40% es más conservadora que la regla del SRSS (Nie, et al., 2010).

Alfredo Reyes Salazar, José Alfredo Juárez Duarte, Arturo López Barraza Juan de Dios Garay Morán y Juan Ignacio Velázquez Dimas evalúan la exactitud de las reglas de combinación de efectos sísmicos SRSS y 30%, utilizando un programa de computo basado en el elemento finito no lineal, en el dominio en el tiempo. Se observa que para las reglas del 30% y del SRSS, el valor del porcentaje λ varía de acuerdo con el periodo de los modelos, del periodo predominante de los sismos, al tipo de respuesta y al tipo de análisis empleado, observando que los valores de λ pueden ser mayores al 30%.

Se encontró para el caso de un análisis elástico, que las reglas del 30% y del SRSS estiman apropiadamente la respuesta combinada en términos de carga axial, no así para el estado inelástico donde se presentan subestimaciones. En el caso de desplazamientos de entrepiso y de cortante basal, las reglas propuestas del 30% y la del SRSS estiman apropiadamente esta respuesta, por lo que respecta a las combinaciones propuestas (40%, 50%, 60% y 1.2Rmax), se observó que presentan resultados muy conservadores (Reyes, et al., 2005).

Antonio B. Rigato, Ricardo A. Medina, examinan la respuesta de estructuras con y sin balance torsional en función del ángulo de incidencia sísmica del movimiento del suelo, variando los grados de inelasticidad y el periodo fundamental de vibración de estas. Se muestra que en promedio las demandas de ductilidad y desplazamiento lateral aplicando diferentes ángulos de incidencia, pueden ser subestimadas hasta un 65%. Lo anterior si se comparan las respuestas obtenidas aplicando el movimiento del suelo en dirección de los ejes principales de las estructuras (principalmente para periodos superiores a 0.5 s). Observaron que las demandas de ductilidad son sensibles al ángulo de incidencia.

El ángulo crítico de una estructura depende del tipo de respuesta deseado, del periodo fundamental y del nivel de comportamiento inelástico de la misma (Rigato & Medina, 2007). En lo que respecta a la determinación de los costos económicos de las estructuras en función de la combinación de los de efectos sísmicos ortogonales, se llevara a cabo una comparación del impacto económico de diseñar una estructura bajo una regla de combinación de efectos sísmicos ortogonales y el nivel de daño de los elementos estructurales.

El costo de las estructuras se evaluará en función del volumen de concreto y del peso del acero de refuerzo de los elementos estructurales. Se pretende que los resultados permitan valorar y determinar la mejor regla de combinación de efectos sísmicos ortogonales para cada diseño de una estructura. Cabe señalar que solamente se cuantificaran volúmenes y costos de los elementos de la superestructura.

A fin de determinar los volúmenes y costos de una estructura, Khaled Alreshaid, Ibrahim M. Mahdi y Ehab Soliman presentan una metodología para optimizar el costo en las estructuras de concreto reforzado, ya que una estructura a base de concreto reforzado representa cerca de un tercio del costo global de una construcción. Mediante una base de datos del mercado de Kuwait, se obtuvieron las relaciones optimas de cero ρ para columnas y vigas, siendo estas de 1.22% y 1.50% de la sección transversal de columnas y trabes respectivamente (Alreshaid, et al., 2004).

R. Izquierdo Ortega e I. Romero Laureani, realizan una comparativa del uso de dos materiales de construcción (concreto reforzado de alta resistencia y acero estructural) para un sistema estructural de 32 niveles de altura en la Ciudad de México. Se presenta el procedimiento de cálculo de volúmenes y costos de los materiales, concluyendo que es ventajoso económicamente el empleo de concreto de alta resistencia frente al sistema de acero estructural (Izquierdo & Romero, 1995).

V.Thiruvengadam, J.C. Wason, Lakshmi Gayathri, elaboran un trabajo de modelado de los costos de estructuras de concreto reforzado, localizadas en varias zonas sísmicas del subcontinente indio. Los elementos que consideran en su análisis son los volúmenes de concreto reforzado, de acero de refuerzo y de cimbrado para el colado de los elementos. La variable más significativa es el acero de refuerzo en vigas y columnas, la cual depende del número de niveles y la zona sísmica en que se ubique la estructura. El costo de una estructura de concreto reforzado al diseñarla de forma resistente a cargas sísmicas, aumenta entre un 2% a 30% dependiendo del número de niveles y de la zona sísmica donde se ubique (Thiruvengadam, et al., 2004).

VII.- Recursos para llevar a cabo la investigación.

Se cuenta con los recursos necesarios tales como equipo de cómputo, software para efectuar el análisis inelástico (programa Canny-e), bibliografía, bases de datos, artículos científicos.

VIII.- Variables.

Características dinámicas y geométricas de los modelos estructurales, ubicación de registros sísmicos, tipo de daño estructural (daño en columnas, vigas etc.).

IX.- Alcances y limitaciones.

1. Se analizará un modelo estructural simétrico (con planta cuadrada haciendo variar las dimensiones de las columnas y trabes a fin de obtener los periodos de la estructura que se deseen). Se describe en la metodología las características generales del modelo.

2. El análisis se efectuará para un rango de periodos entre 0.3 y 2.5 segundos, rango crítico para estructuras ubicadas en terrenos de tipo duro, de transición y blandos

3. No se considerará el efecto de la componente vertical del movimiento del suelo en la respuesta de los modelos.

4. No se considera en el modelo desarrollado la interacción suelo estructura.

X.- Metodología.

1. Revisión bibliográfica: Se revisarán los trabajos de investigación relacionados directamente con el tema propuesto. Se consultará la bibliografía para definir y ampliar el conocimiento de los temas principales de este trabajo.

2. Antecedentes y marco teórico: En esta parte se describirán los trabajos relacionados con el tema de investigación, mismos que dan soporte y orientación al trabajo. Se definirán los conceptos y términos que estén contenidos en la investigación a fin de dar mayor comprensión y referencia del tema.

3. Planteamiento del modelo estructural y parámetros de análisis: se definirán las propiedades geométricas y dinámicas del modelo estructural a analizar. Así mismo se definirán las condiciones del suelo y las características de los sismos a emplear.

4. Pruebas del modelo: mediante el uso de software se realizarán corridas del modelo estructural bajo las condiciones definidas en el punto tercero de la metodología y utilizando una regla de combinación de efectos sísmicos ortogonales.

5. Análisis de resultados: mediante los resultados del análisis, se observará la variación de las respuestas en la estructura, con lo cual se establecerán parámetros que relacionen dicha respuesta con las propiedades geométricas, dinámicas y la combinación de efectos sísmicos ortogonales de una estructura.

6. Reporte de resultados: Se mostrarán de manera resumida los resultados determinados por la investigación. Se darán recomendaciones acerca del diseño y costo de las estructuras en función de la combinación de efectos sísmicos ortogonales.

7. Elaboración de artículo: presentar el trabajo de investigación mediante un artículo científico a fin de ser enviado a una revista indexada, para su publicación.

XI. Cronograma

Actividad/Tiempo	Semestre			
	Primero	Segundo	Tercero	Cuarto
Revisión Bibliográfica				
Antecedentes y Marco Teórico				
Planteamiento de modelos de estudio y selección de acelerogramas.				
Pruebas del modelo				
Análisis inelástico de modelos (Canny)				
Análisis de resultados				
Reporte de resultados				
Elaboración de articulo.				

XII. - Bibliografía y referencias.

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PARTE 2

ARTÍCULO DE INVESTIGACIÓN

Engineering Structures <EviseSupport@elsevier.com>
Jue 28/11/2019 14:27, Mejia Perez Daniel

Dear Mr. Mejía-Pérez,

You have been listed as a Co-Author of the following submission:

Journal: Engineering Structures

Title: Assessment of the inelastic structural response for building models considering the combination of the orthogonal seismic effects

Corresponding Author: JESUS VALDES-GONZALEZ

Co-Authors: Jaime De-la-Colina, Daniel Mejía-Pérez

JESUS VALDES-GONZALEZ submitted this manuscript via Elsevier's online submission system, EVISE®. If you are not already registered in EVISE®, please take a moment to set up an author account by navigating to http://www.evise.com/evise/faces/pages/navigation/NavController.jspx?JRNL_ACR=ENGSTRUCT

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If you did not co-author this submission, please contact the Corresponding Author directly at jvaldes@uaemex.mx.

Thank you,

Engineering Structures

Manuscript Details

Manuscript number	ENGSTRUCT_2019_4926
Title	Assessment of the inelastic structural response for building models considering the combination of the orthogonal seismic effects
Article type	Research Paper

Abstract

The inelastic behavior of several structural models with different translation periods in both orthogonal directions was studied. For each model the incidence angle of the earthquake was varied from 0° to 90 ° with increments of 10°. The combination of the horizontal orthogonal seismic effects used for design varied for α = 10%, 30%, 50%, 70% and 100%. Twelve seismic records, 6 of firm soil and 6 of soft soil, were analyzed. Two reduction factors of design elastic forces were used (Q = 2, and 4). Based on these parameters, different analyses were carried out. For each one, the ductility demands of columns and beams were evaluated, as well as interstory drifts. With these interstory drifts, the damage of the models was estimated for each case. The total cost of the models was evaluated, considering the building initial cost, repair costs, content losses, income loss costs, and losses due to injured and deceased people. The principal factor that affects the total cost of the models was the type of soil, however the other variables also modify the estimation of the total cost. For firm soil, the average of the optimum α is 0.85 for Q = 2 and 0.9 for Q = 4. For soft soil the optimum α average is 0.20 and 0.30 for Q = 4. The ductility demands also strongly vary as a function of the studied variables. The main variable that affects ductility demands is the incidence angle. The α values that approximate to a greater extent the computed ductility demands with the Q values were as follows: for firm soil, 0.1 for Q = 2 and 0.5 for Q = 4; for soft soil, 1.0 for Q = 2 and 0.7 for Q = 4.

Keywords	orthogonal seismic effects; combination rules; inelastic response; damage costs; damage index
Taxonomy	Structural Engineering, Earthquake Engineering, Building Design, Earthquake Effect on Structures
Manuscript region of origin	North America

Corresponding Author	JESUS VALDES-GONZALEZ
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Suggested reviewers	Mario Ordaz, Konstantinos Kostinakis, Antonio B. Rigato

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Universidad Autónoma del Estado de México
UAEM

November 28, 2019.

Dr. P.L. Gould
Editor-in-Chief Engineering Structures

Dear Dr. Gould:

We are sending to you the manuscript entitled: "Assessment of the inelastic structural response for building models considering the combination of the orthogonal seismic effects" which we are submitting for exclusive consideration of publication as a research paper in Engineering Structures.

The paper shows the results of an analytical study conducted to evaluate the influence of the selected percentage in the orthogonal seismic effects combination rules. This evaluation is done by mean of the analysis of several building models with inelastic behavior.

The study considers the influence of different variables in the behavior of the models, such as: soil characteristics, earthquake record, earthquake incidence angle, percentage of combination rules, translation lateral periods of the models and reduction factor for elastic design forces.

Considering all possible values for these variables, the ductility demands and total costs of the models are computed.

In accordance with the results, different percentages to combine the orthogonal seismic effects are suggested with the purpose minimize building total costs and to obtain ductility demands close to the design ductility.

Thank you very much for your consideration.

Sincerely,
The authors
Daniel Mejía Pérez
Jesús Valdés González
Jaime De la Colina Martínez

Assessment of the inelastic structural response for building models considering the combination of the orthogonal seismic effects

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HIGHLIGHTS

Combination of horizontal orthogonal seismic effects on building models.

Total costs evaluation of inelastic building models.

Ductility demands evaluation of inelastic building models.

Behavior of inelastic building models for different soil characteristics

Influence of incident angle in the behavior of inelastic building models

Optimal percentages to combine the orthogonal seismic effects on buildings

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Assessment of the inelastic structural response for building models considering the combination of the orthogonal seismic effects

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ABSTRACT

The inelastic behavior of several structural models with different translation periods in both orthogonal directions was studied. For each model the incidence angle of the earthquake was varied from 0° to 90 ° with increments of 10°. The combination of the horizontal orthogonal seismic effects used for design varied for α = 10%, 30%, 50%, 70% and 100%. Twelve seismic records, 6 of firm soil and 6 of soft soil, were analyzed. Two reduction factors of design elastic forces were used (Q = 2, and 4). Based on these parameters, different analyses were carried out. For each one, the ductility demands of columns and beams were evaluated, as well as interstory drifts. With these interstory drifts, the damage of the models was estimated for each case. The total cost of the models was evaluated, considering the building initial cost, repair costs, content losses, income loss costs, and losses due to injured and deceased people. The principal factor that affects the total cost of the models was the type of soil, however the other variables also modify the estimation of the total cost. For firm soil, the average of the optimum α is 0.85 for Q = 2 and 0.9 for Q = 4. For soft soil the optimum α average is 0.20 for Q = 2 and 0.30 for Q = 4. The ductility demands also strongly vary as a function of the studied variables. The main variable that affects ductility demands is the incidence angle. The α values that approximate to a greater extent the computed ductility demands with the Q values were as follows: for firm soil, 0.1 for Q = 2 and 0.5 for Q = 4; for soft soil, 1.0 for Q = 2 and 0.7 for Q = 4.

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Keywords: orthogonal seismic effects; combination rules; inelastic response; damage costs; damage index.

1. INTRODUCTION

The seismic ground motion has six components, three of translation and three of rotation. However, given the way the ground movement is recorded, only two horizontal orthogonal translation components are usually considered. If the structure is located near the epicenter of the earthquake, the vertical component must also be included [1].

The conventional analysis of structures is carried out individually for each of the components of the seismic movement that is relevant. Subsequently, the maximum responses of each component are combined to obtain estimations of the maximum response that would result when all components of the ground movement act simultaneously [1]. Therefore, most of the design specifications and recommendations use combination rules to incorporate the effects of the main ground motion components.

The most common used rule to combine the orthogonal seismic effects, it is the so called percentage combination rule, which computes the structural response as the sum of 100% of the effects caused by the action of one earthquake component acting along one direction of the structure plus a α percentage of the response caused by the action of the another earthquake orthogonal component acting along the orthogonal axis of the model. Which it is represented through Eq. (1).

$$\text{Maximum response} \cong max \begin{Bmatrix} 100\% S_x + \propto S_y \\ 100\% S_y + \propto S_x \end{Bmatrix} \quad (1)$$

where S_x is the response of the structure due to the action of the earthquake component along the X direction and S_y the response due to the action of the earthquake component along the Y direction.

It is assumed for design purposes, that the horizontal orthogonal components of the earthquake ground motion act along the direction of the structure principal axes; however the incidence angle of the earthquake is uncertain. It has been showed that such orientation changes the maximum response of a structure. Not explicitly consider the critical orientation for the earthquake incidence angle can significantly underestimate the maximum response [2].

Rosenblueth and Contreras [3] proposed a value of the percentage for seismic combination of α=30%. On the other hand, Newmark [4] proposed a value of α=40%. Additionally, some codes specify a third rule, which estimates the maximum bidirectional response as the square root of the sum of the maximum unidirectional responses squared (SRSS). Smeby and Der Kiureghian [5], Menun and Der Kiureghian [6] proposed CQC3 rule, which explicitly takes into account the correlation between modal responses and horizontal components of ground motion. López and Torres [7] developed an equation that allows to obtain the angle of incidence that leads to the maximum response.

Reyes *et al*. [8] indicate that the maximum response of a structure is not always presented discomposing the earthquake ground motion along the structure axes defined by its geometry in plan. Moreover, the critical incidence angle varies from one earthquake to another and depends on the type of structural system, the location of the structural member and the level of deformation of the structure. On the other hand, Hosseini et al. [9] observed that for a structure, the critical response of its structural elements occurs for different angles and varies from one earthquake to another; so there is no fixed angle that estimates the maximum responses for all structural elements. Rigato and Medina [10], show that on average the demands of ductility and lateral displacement in a structure can be underestimated up to 65% depending on the incidence angle of the earthquake. The critical angle of a structure depends on the type of response (axial load, interstory drift, etc.), the fundamental period of vibration and the level of inelastic behavior thereof.

Valdés et al. [11] developed some analytical expressions to estimate the maximum response of elastic structures considering earthquake records in soft soil. These expressions explicitly considers the angle of the earthquake incidence and the type of response in terms of the direction of its components (orthogonal or collinear). Kostinakis et al. [12] concluded that the 30% and 40% rules underestimate the maximum response obtained for all angles of incidence in a structure. Shahryari *et al*. [13], proposed values of the combination parameter α, depending on the geometric shape of the structure plan and the structure lateral load resistant system. Potnis *et al*. [14] proposed a method that conduct to low and acceptable underestimation levels in order to obtain the maximum response for any orientation of the structure axes.

Heredia and Machicao [15] observed that the effect of the horizontal components of the earthquake is different depending on the structural system and the natural period of vibration of the system. They pointed out that the combination rules (30% and 40%) can be very conservative or underestimate the dynamic response, since they do not consider the properties of the structure, nor the soil conditions in which they are located. López *et al*. [16] determined that the simplified SRSS, 30% and 40% rules overestimate the critical response. They also indicated that the critical response increases when the main modal periods are close to each other.

Reyes et al. [17] observed that the value of the percentage of seismic combination α varies according to the period of the structural models, the predominant period of the ground motion, the type of response of the structure and the type of analysis used, they observed that the value of α can be greater than 30%.

The rules for combining orthogonal seismic effects 100% + 30% and 100% + 40% are the most common that are specified in the recommendations and design codes [18, 19, 20, 21, 22, 23].

2. PROBLEM STATEMENT

The design codes [18, 19, 20, 21, 22, 23] specify different rules to combine the orthogonal seismic effects, there is not a clear consensus regarding the implications of the use of each recommendation. On the other hand, several studies have showed that these rules have several limitations. Among these limitations are: a) they were developed from elastic analyzes of structures, leaving aside their nonlinear behavior, b) they do not consider the fundamental periods of vibration of the structure, c) they do not consider the type of soil on which the structures are located and d) they do not explicitly consider the critical angle of rotation with which the axes of the structure must be oriented to get the maximum responses [10].

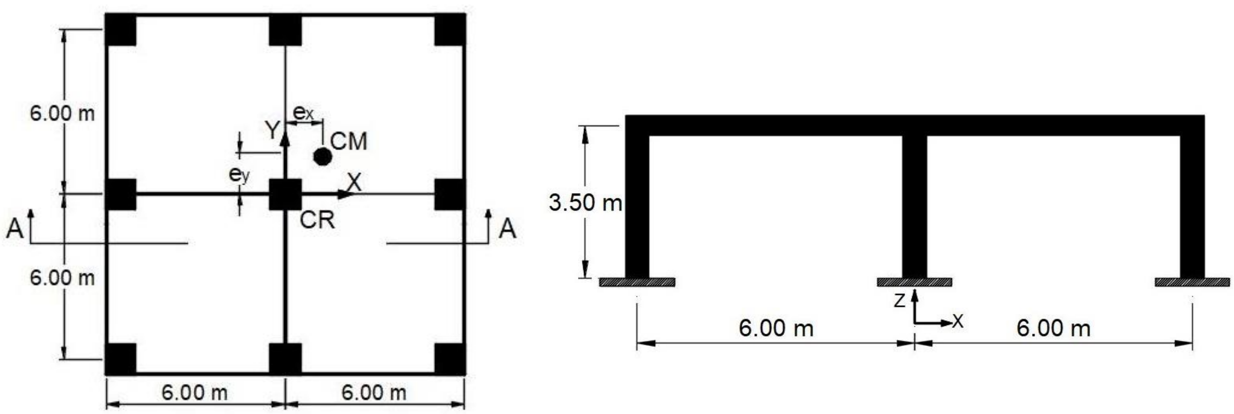
The aforementioned works have studied some aspects related to the rules of combination of orthogonal seismic effects, however, there are not studies related with total costs of building that include construction costs and economic losses due to earthquake damages. In this work, total costs of buildings are computed based on the α value used. In addition, there is no general consensus regarding the value of the percentage of orthogonal seismic effects that should be used for each type of structure. On the other hand, there is a notable absence of nonlinear studies that explicitly consider the effect of α on the response of a structure. In particular, it is interesting to know the inelastic behavior of buildings designed under different α values as a function of their dynamic properties and the type of soil. In this work, representative building models with different dynamic properties are studied for firm and soft soil.

In this work different three-dimensional building models representative of reinforced concrete buildings are analyzed, which have different fundamental periods of translational vibration in their orthogonal directions. The inelastic response of the models is calculated considering different values of α in their design, which are: 10%, 30%, 50%, 70% and 100%, (in order to avoid confusion, in the text the α values are expressed in decimal form, α = 0.1, 0.3, 0.5, 0.7, 1.0).

From these analyzes, the ductility demands of columns and beams of the models are obtained, as well as the lateral drifts. Based on lateral drifts, the structural damage of the model is estimated. In the analyzes, different rotation angles of the structure axes are considered (incident angle). Taking into account the aforementioned variables, as well as, the costs of construction and the economic losses due to earthquake damage, it is possible to evaluate the behavior of each structural model for the different α values studied.

3. STRUCTURAL MODEL

The studied base structural model is a three-dimensional single-story structure with the geometry showed in Fig. 1. Center of mass eccentricities in both orthogonal directions equal to 5% of the perpendicular dimension in plan are considered. This eccentricity value is according to different design regulations [20, 23]. The model has three degrees of freedom: two translation displacements along the X and Y directions, and a rotation around the Z axis. The columns of the model were considered fixed in the base.



a) Plan view of structural model b) Lateral view A-A

Figure 1. Plan and lateral view of structural model

The base structural model was modified to generate 30 models (15 for soft soil and 15 for firm soil) which are differentiated by their lateral periods. For the firm-soil models the following periods were analyzed: 0.4, 0.5, 0.7, 0.8 and 0.9 s, while for the soft-soil models the periods were 1.4, 1.7, 2.0, 2.3 and 2.5 s. The stiffness of the base varied by modifying the dimensions of columns and beams in order to get the different lateral periods. The stiffness center was located in the geometric center of the model. In total, 30 different models are analyzed, which result from the different combinations of lateral periods for X and Y directions. The selection of the studied model periods was done to match their values with the main periods contained in the considered earthquake records. A 5% damping is used for all models.

A spectral analysis was carried out considering both components of the ground motion for each seismic record and for each angle of incidence. For each case the response spectrum was taken as the design spectrum. The columns were designed to biaxial flexural-compression considering ratios of steel area / concrete area from 1% to 4%, while the beams were designed to flexural action considering maximum ratios of steel area / concrete area of 1.0%. Under these considerations and for each value of α, the design of columns and beams was carried out according to the hypotheses indicated in the ACI 318-19 [24], considering load factors and reduction strength equal to 1.0. These considerations were followed to prevent the results from being affected by variables other than those related to this study.

For all models, two types of beams are identified: edge (BB) and interior (BI), while, for columns, three types are identified: edge columns (CB), corner columns (CC) and interior column (CI), Fig. 2. All columns and beams of the same type had the same longitudinal reinforcement. Two different values for de design ductility demand were used in the study, Q = 2 and Q = 4. For the purposes of this study, the Q factor only takes into account the reduction by ductility of the seismic forces. The Q factor is equivalent to the R factor used in the IBC code [25].

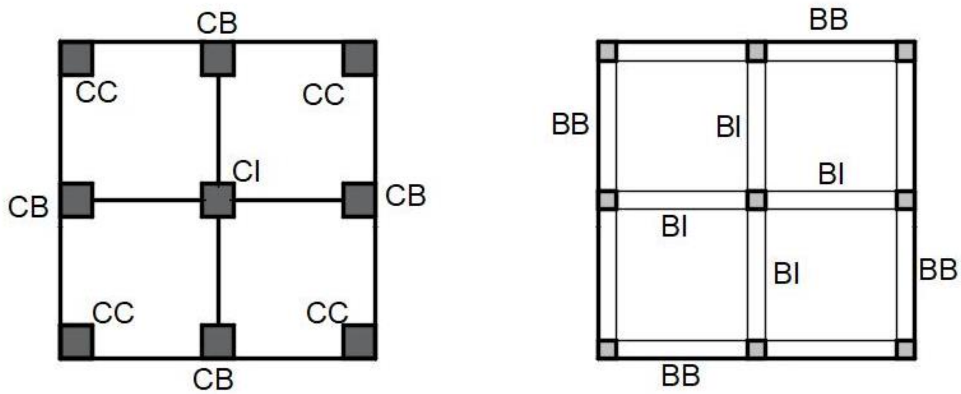


Figure 2. Identification of structural elements

The studied models do not correspond to any particular actual building, they are abstracts models which purpose is to represent buildings with dynamics properties similar to those of them. In particular, it is interesting to analyze the inelastic dynamic behavior of models that have certain combinations of lateral periods. The physical properties of the models allowed to obtain the lateral periods of interest. Several studies have made use of generic models similar to those used in this work [10, 11, 15].

4. INCIDENCE ANGLE AND COMBINATIONS OF ORTOGONAL SEISMIC EFFECTS

To obtain the maximum response, it is necessary to analyze the models for different ground-motion incidence angles. The orientation of the ground motion was varied in increments of 10°, from 0° to 90°. Thus, for each seismic record there were 10 cases of analysis, one for each angle of incidence. The columns and beams were designed for the envelope of the combinations of orthogonal seismic effects that appear in Table 1.

Table 1. Design combinations

1	$100\%S_x + \alpha S_y$	5	$\alpha S_x + 100\%S_y$
2	$100\%S_x - \alpha S_y$	6	$\alpha S_x - 100\%S_y$
3	$-100\%S_x + \alpha S_y$	7	$-\alpha S_x + 100\%S_y$
4	$-100\%S_x - \alpha S_y$	8	$-\alpha S_x - 100\%S_y$

In total, 12 pairs of seismic records are considered, 6 correspond to firm soil and 6 to soft soil. A total of 180 different scaling coefficients were used (one for each model and for each earthquake). This scaling was carried out so that the design of columns and beams corresponded to the ratios of steel area to concrete area established in the section 3 of the paper. Some characteristics of these seismic records are presented in tables 2 and 3.

Table 2. Seismic records on firm soil

Station	Place	Year	Magnitude	PGA (cm/s^2)
Imperial Valley	United States	1940	6.9	331.0
Javier Barros	Mexico	2017	7.1	92.1
Montenegro	Yugoslavia	1979	6.9	64.6
Valparaíso	Chile	1985	7.8	303.4
Loma Prieta	United States	1989	7.0	613.6
Ciudad Universitaria (CU)	Mexico	1985	8.1	33.2

Table 3. Seismic records on soft soil

Station	Place	Year	Magnitude	PGA (cm/s^2)
Bucharest	Rumania	1977	7.2	169.2
Sylmar-Northridge	United States	1994	6.7	569.1
SCT	Mexico	1985	8.1	161.6
SCT	Mexico	1999	7.0	31.3
SCT	Mexico	2017	7.1	91.1
Tlatelolco	Mexico	2017	7.1	85.5

5. COSTS EVALUATION

It is essential to consider the economic losses caused by an earthquake to buildings. The total cost of a building (C_E) can be obtained through Eq. 2 [26]:

$$C_E = C_I + C_D \tag{2}$$

where C_I is the initial cost of the building, which mainly involves the construction cost, C_D is the cost in present value of the damage caused to the building by the earthquake, which can be calculated using Eq. 3 [26]:

$$C_D = C_R + C_C + C_E + C_H + C_M \tag{3}$$

where C_R is the cost of repair or replacement, C_C is the cost of contents, C_E is the cost due to the loss of occupation and income of the building, C_H is the cost due to the injured people and C_M is the cost due to the deceased people during the earthquake. For each case (each model, each α, each ground motion, each angle of incidence and each Q) both, cost and ductility demands, were computed.

The equations proposed by Ang and De León [26] use of a damage index D, which varies between 0 and 1 (0 corresponds to zero damage and 1 to extreme damage and / or collapse of the structure). These authors recommend a value of D = 0.5, as the maximum damage index to repair a structure. The level of damage D in the model is determined through the maximum value of the interstory drift. According to Ghobarah [27], the damage level can be associated with the maximum interstory drift (Table 4).

Table 4. Relation between damage level and interstory drift (Ghobarah 2004)

No.	**Damage level**	**Drift, Q = 2**	**Drift, Q = 4**	**Damage index (D)**
1	No damage	<0.10	<0.20	0.10
2	Repairable Damage (Light)	>=0.10, <0.20	>=0.20, <0.40	0.15
3	Repairable Damage (medium)	>=0.20, <0.50	>=0.40, <1.0	0.20
4	Irreparable damage	>=0.50, <0.90	>=1.0, <1.80	0.40
5	Severe damage (partial collapses)	>=0.90, <1.5	>=1.80, <3.0	0.60
6	Total collapses	<1.5	<3.0	1.00

Ang and De León [26] proposed to evaluate the costs with the following considerations: The repair cost depends on the damage index and the replacement cost of the original building (C_r). It is given by Eq. 4.

$$C_R = 1.64 C_r D; \quad 0 \leq D < 0.5 \tag{4}$$

The replacement cost of the original building C_r is considered constant for all types of buildings. This cost generally involves the economic cost of demolition, rubble removing and rebuilding. The value of C_r is considered equal to 1.40 times the building initial cost, which is obtained from the ratio between the repair costs indicated by FEMA 227 [28] and the construction costs indicated by Popescu et al. [29].

The cost for the loss of content (C_c) is calculated with Eq. 5 [26], which assumes that the loss of content can represent up to 50% of the value of the replacement cost of the original structure.

$$C_c = 0.5\, C_r D \tag{5}$$

Regarding the cost due to the loss of occupation and income of the building (C_E) , the proposal made by [26] is taken as a basis. In this case, these costs are calculated by Eq. 6.

$$C_E = 0.15\, C_r D^2 \tag{6}$$

where the coefficient of 0.15 is obtained through the ratio of the average annual rent cost of a building between its construction cost per square meter. Similarly, the costs for injured people (C_H) and dead people (C_M), are taken from Ang and De León [26]. Costs for injured people are calculated by Eq. 7.

$$C_H = 0.26\, T_H C_r D^2 \tag{7}$$

where the coefficient of 0.26 is obtained by dividing the average cost of hospitalization of a patient by the cost of building construction per occupant and T_H is the expected rate of injured people in accordance with the level of damage to the building [28].

The cost due to deceased people is obtained through Eq. 8 [25]. In this case the coefficient of 255 is obtained by dividing the cost per deceased person by the construction cost of the building per occupant. T_M is the expected rate of deceased people based on the level of damage of the building [28].

$$C_M = 255\, T_M C_r D^4 \tag{8}$$

The initial cost of each structural model was obtained by quantifying the of materials (concrete and steel) that resulted from the design of each of model. These costs included materials, tools and workforce. In this work it is proposed, that the cost of the structure of a building represents on average 35% of the building total cost (which includes: partition walls, installations, finishes, etc.). It is important to note that the costs of repair or replacement of the buildings in this work correspond to the money present value. Moreover, it is also important to bear in mind that the building costs computation was done by assuming a full certainty in the occurrence of the design earthquake.

6. ANALISYS OF RESULTS

Initially, as an example, two structural models are used to show the influence of the incidence angle and the earthquake record. The first model (firm soil) has T_x = 0.5 s and T_y = 0.7 s, while the second model (soft soil) has T_x = 1.7 s and T_y = 2.3 s. Subsequently, the results of ductility demands and total costs of all the models are showed. A total of 18,000 cases of analysis were performed, obtained through 30 models (15 for firm soil and 15 for soft soil), 6 seismic records for each one of them, 10 angles of incidence, 5 values of α and 2 values of Q. Each model was analyzed inelastically with the Canny-e program [30], obtaining the ductility demands μ of beams and columns, as well as the interstory drift. The interstory drift helped to calculate the damage index for the evaluation of the total cost of the structural models.

6.1 Influence of the ground motion incidence-angle

Several incidence angles of the orthogonal components of earthquake ground motion for the model with T_x= 0.5 s and T_y = 0.7 s are analyzed in this section. An important variation was observed in the ductility demands μ of columns and beams. Fig. 3 shows the ductility demands of columns. The horizontal axis contains the incidence angle and the vertical axis shows the ductility demands of all columns. Fig. 4 shows the maximum and minimum values of ductility demands of columns and beams for the structural model with T_x = 1.7 s and T_y = 2.3 s. The horizontal axis contains the analyzed α values, while the vertical axis contains the ductility demand. The maximum demand ductility values correspond with the critical incidence angle of the ground motion.

For Q = 2, the columns of located model on soft soil show greater variation of their ductility demands in comparison with the model corresponding to firm soil. This variation is 15% greater for the soft soil model than for firm soil model. For Q = 4, the behavior is reversed, in this case, the model in firm soil shows a greater variation of columns ductility demands. For both models, soft and firm soil, the interior column (CI) has the minimum ductility demand variation. In the case of beams, for both Q = 2 and Q = 4, model on soft soil show greater variation in the ductility demands. This variation is 12% greater for the soft soil model than for the firm soil model.

Next, the results for all models and analysis cases are showed. Tables 5 and 6 show the averages of minimum and maximum values of ductility demands ($\bar{\mu}$) in the structural elements of all the models of firm and soft soil. These results consider all angles of incidence, all seismic records, all α values and both Q values. The coefficient of variation (C.V.) of ductility demands is included.

Table 5. Averages of minimum and maximum values of ductility demands for firm soil

Element	Mean minimum value				Mean maximum value			
	Q = 2		Q = 4		Q = 2		Q = 4	
	$\bar{\mu}$	C.V.	$\bar{\mu}$	C.V.	$\bar{\mu}$	C.V.	$\bar{\mu}$	C.V.
CC	**0.90**	0.13	3.20	0.15	2.62	0.14	5.40	0.11
CB	1.00	0.14	**2.23**	0.16	**2.80**	0.13	**5.80**	0.11
CI	1.26	0.15	2.47	0.14	1.96	0.09	4.20	0.10
BB	1.40	0.18	**2.33**	0.27	2.71	0.20	4.72	0.22
BI	**1.10**	0.13	3.10	0.19	**3.30**	0.22	**5.00**	0.22

Table 6. Averages of minimum and maximum values of ductility demands for soft soil

Element	Mean minimum value				Mean maximum value			
	Q = 2		Q = 4		Q = 2		Q = 4	
	$\bar{\mu}$	C.V.	$\bar{\mu}$	C.V.	$\bar{\mu}$	C.V.	$\bar{\mu}$	C.V.
CC	**1.20**	0.42	2.80	0.43	3.10	0.18	5.05	0.05
CB	1.40	0.52	**2.34**	0.40	**3.30**	0.16	**5.90**	0.06
CI	1.56	0.40	3.30	0.42	2.92	0.14	4.93	0.09
BB	1.60	0.45	3.00	0.39	2.68	0.39	4.69	0.26
BI	**1.07**	0.44	**2.31**	0.45	**3.20**	0.37	**5.10**	0.30

Tables 5 and 6 show that models on soft soil have the highest averages of ductility demands on columns. The highest ductility demand for columns in soft-soil models is 5.9 for Q = 4, and 3.3 for Q = 2. For models on firm soil, the maximum ductility on columns is 5.8 for Q = 4 and 2.8 for Q = 2. Column CI, in both soils, firm and soft, has the lower values of ductility demands averages compared with columns CC and CB. The ductility demands averages on beams BB and BI have similar values for the two types of soil studied. The maximum ductility demand average on beams are 5.1 for Q = 4 and 3.2 for Q = 2. In general, it is observed that the variation coefficient of ductility demands on columns and beams is greater for models on soft soil than for models on firm soil.

In general, the incidence angle of the earthquake is the main factor that influences the variation of the structural response of the models. As it has been showed, important variations in the ductility demands of the members of a structure can be presented. Considering all α values, the incidence angle can generate ductility demands that exceed the Q value in 40% for firm-soil models and 60% for soft-soil models.

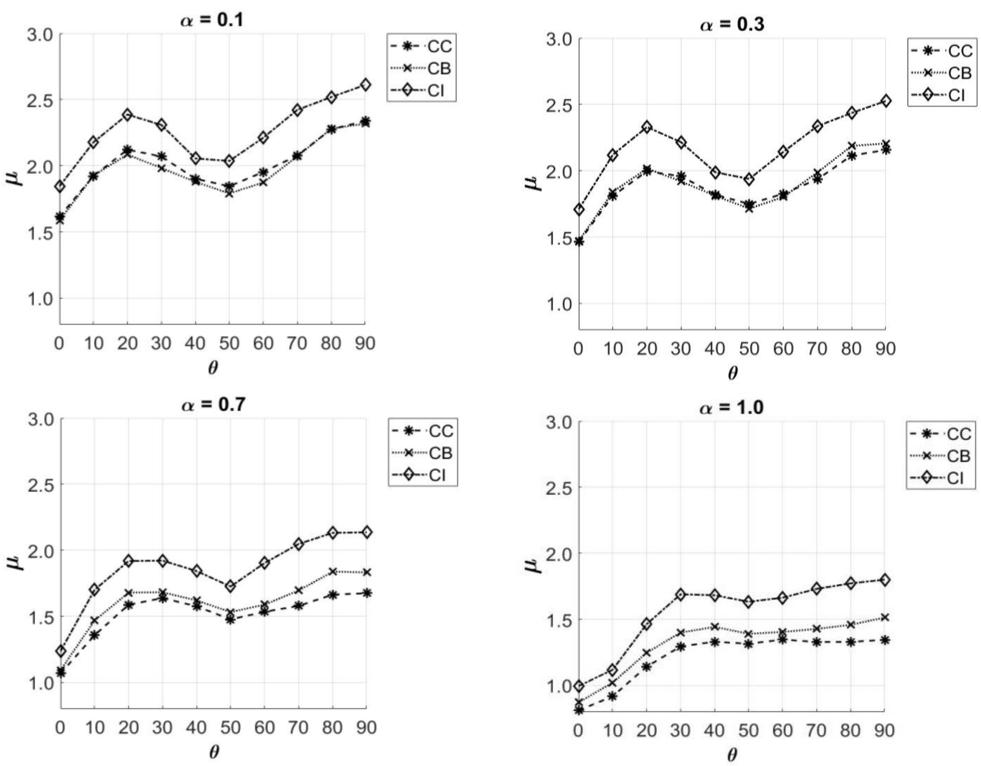


Figure 3. Variation in ductility demands of the model with T_x = 0.5 s and T_y = 0.7 s, designed with Q = 2 (firm soil)

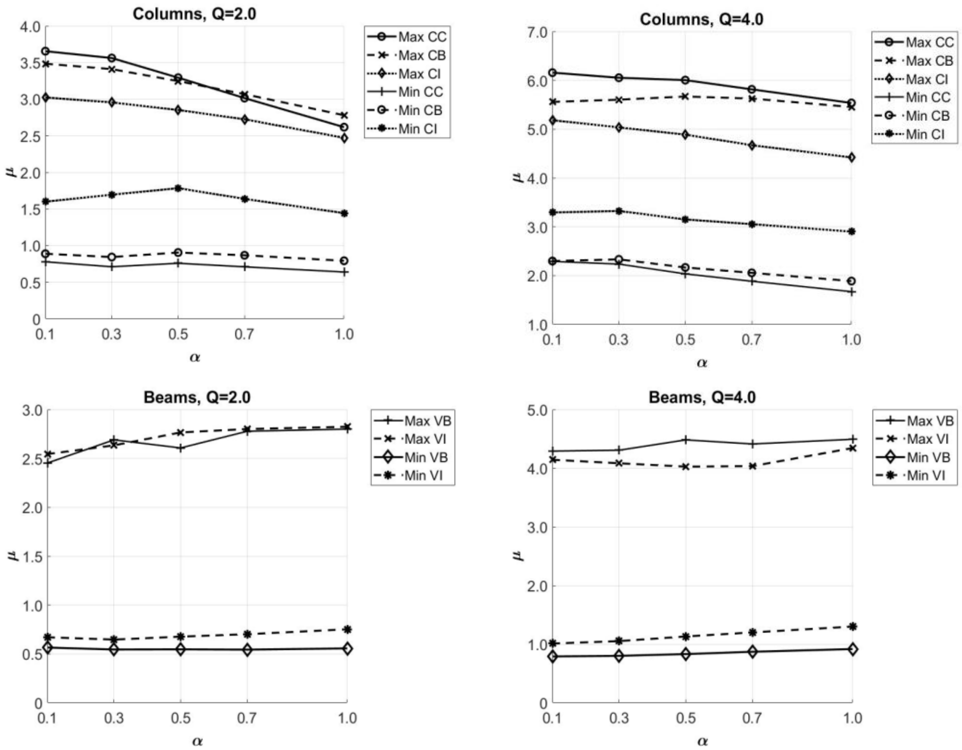


Figure 4. Maximum and minimum ductility demands, model T_x = 1.7 s and T_y = 2.3 s (soft soil)

6.2 Earthquake Influence

The maximum variation in the ductility demands of the corner columns and edge beams of the model (soft soil) with periods T_x =1.7 s and T_y = 2.3 s for all different earthquakes are showed in Fig. 5. The horizontal axis of the graphs presents the values of α, while the vertical axis shows the ductility values (which considers all the angles of incidence analyzed). In soft soil, the variation in ductility demands of columns for the maximum and minimum values of all analyzed earthquakes is 40% for Q = 2 and 25% for Q = 4.

For firm soil, the variation in ductility demands of columns for the maximum and minimum values of all analyzed earthquakes is 45% for Q = 2 and 30% for Q = 4. In the case of beams, for the two types of soil and for the two Q values studied, the variation in ductility demands for all earthquakes analyzed is not greater than 10%. It is important to note that the obtained results correspond to the average of all earthquakes. However, for some earthquakes studied, the ductility demands of columns and beams reach values greater than the Q value. These values correspond to the percentages indicated in the section where the incidence angle is analyzed (Section 6.1).

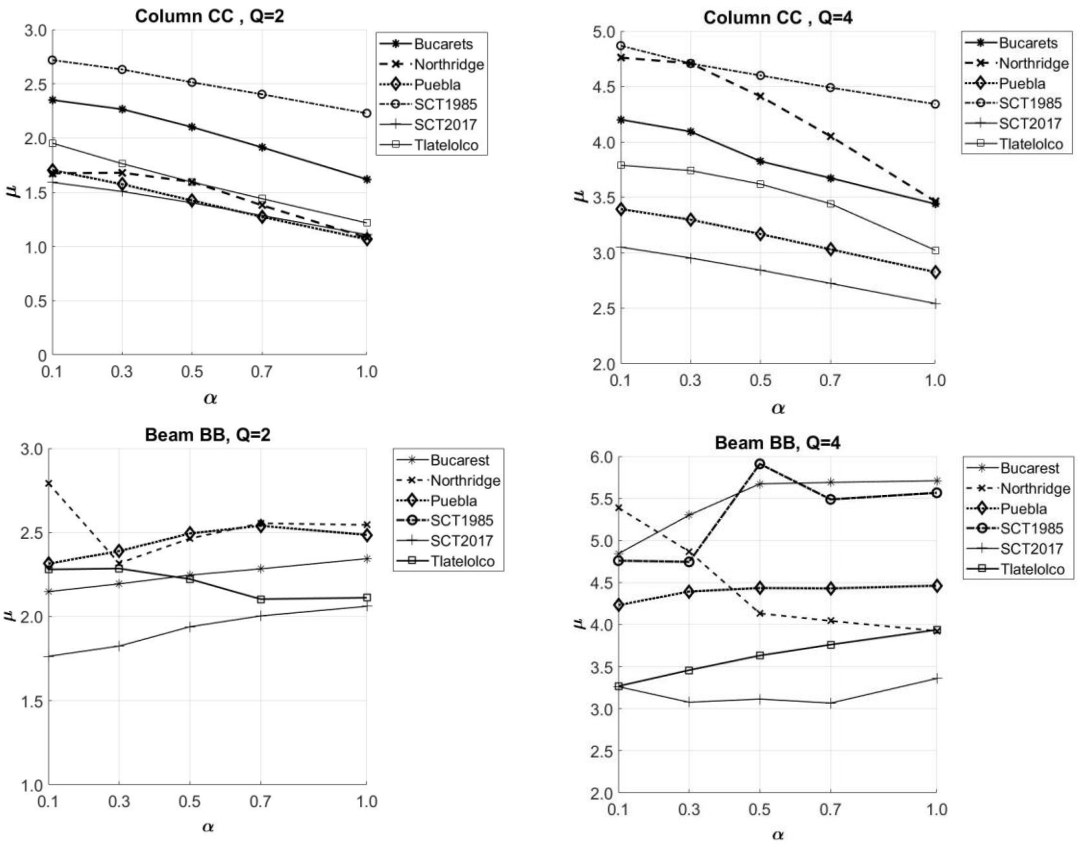


Figure 5. Ductility demands according to the earthquake, model $T_x = 1.7$ s and $T_y = 2.3$ s

6.3 Total damage cost

Figure 6 shows the different costs that integrate the total damage cost corresponding to one of the studied models for a specific earthquake. The horizontal axis contains the α values while the vertical axis contains different damage costs normalized with respect to the initial cost. The * denotes the normalization done.

The graph corresponding to the upper left corner of figure 6 shows the behavior of the damage index D as a function of α. It is noted how the damage index decrease to the extent that α values increase. This is because the structural elements are more reinforced, and in general, the models result more resistant to the extent that high α values are used in their design.

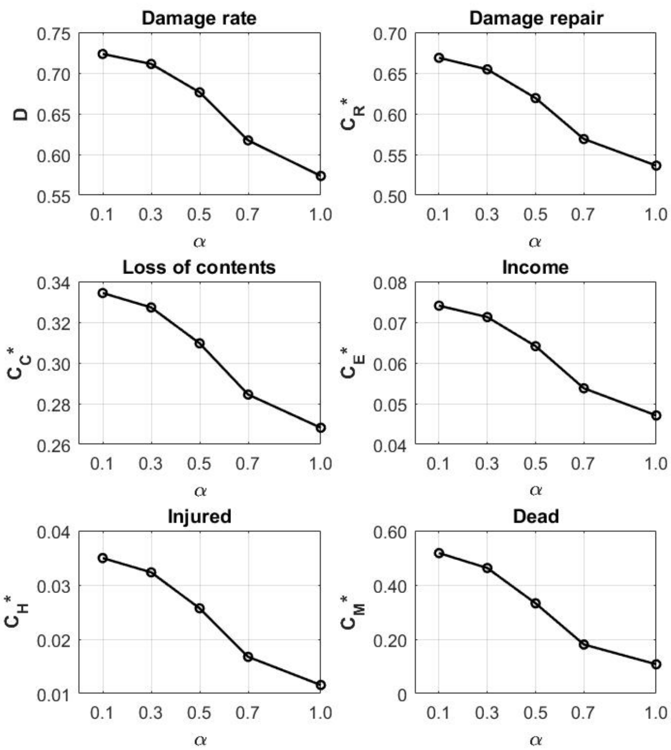


Figure 6. Damage index and normalized damage costs for model with T_x = 0.5 s and T_y = 0.7 s

Figure 6 shows, for this particular model, that the higher costs correspond to the reparation of the building and fatalities. It is observed that for α = 1.0, in which case the structure is more strengthened, all costs are minor. In general, structures designed with high α values have a high initial cost, but low costs for damages.

Figures 7a and 7b show the total cost of two analyzed models for a specific seismic event (1985 México earthquake recorded in CU (firm soil)). The horizontal axis contains the α values, while in the vertical axis contains the total cost of the building normalized with respect to its initial cost (C_E*). Fig. 7a corresponds to the model with T_x = 0.5 s and T_y = 0.7 s, while fig. 7b corresponds to the model with T_x = 1.4 s and T_y =

2.3 s. It is observed as the α optimum value that minimizes the total cost varies from one model to another. It can be seen that the value of optimal α (minimum cost) varies from one model to another. In this case, the optimum value is α = 1.0 for one of the models (fig. 7a) and α = 0.3 for another model (fig. 7b).

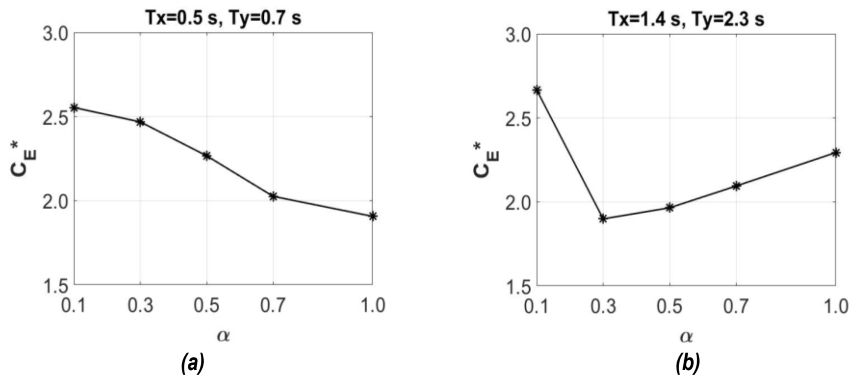


Figure 7. Total cost of two models corresponding to CU 1985 México earthquake (firm soil)

Fig. 8 shows the total costs for the model with T_x = 1.4 s and T_y = 2.3 s considering all earthquakes. The horizontal axis shows α values and the vertical the total cost normalized with respect to its initial cost (C_E*). Also, all the incidence angles for the earthquakes are considered in the computations. It is appreciated that the optimum α values vary from one earthquake to another for the same model, even for the same earthquake considering different values of Q.

The maximum, minimum and average values of C_E* are 2.85, 1.60 and 2.20 respectively. It is noticed that the type of earthquake is a relevant variable in determining the optimal α value.

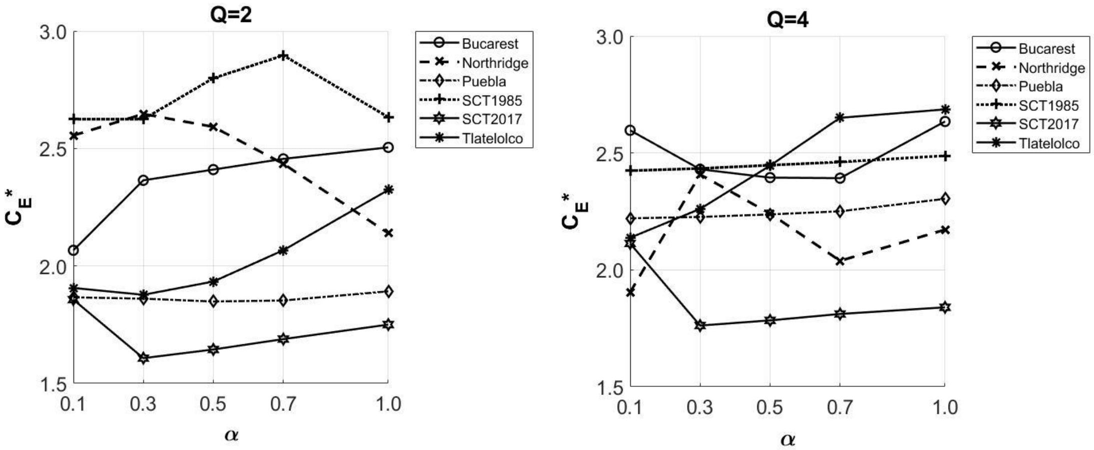


Figure 8. Ratio between total cost and initial cost, model Tx = 1.4 s and Ty = 2.3 s.

6.4 Ductility demands of all models

Figures 9.1, 9.2, 9.3 and 9.4 show the results of maximum ductility demands for corner columns and edge beams of firm-soil models considering all different earthquake records, incidence angles and α values. The horizontal axis shows the translation period in X direction of the models and the vertical axis the ductility demands. Different plots in each graph correspond to translation periods of models in Y direction. The behavior of edge columns is similar to that of corner columns (figs. 9.1 and 9.2). The behavior of central columns exhibit a trend to maximum ductility demands close to that used in the corresponding design (Q =2, Q = 4). For the central column, the variation in ductility demands is less than 40% compared with the edge and corner columns. For models with equal periods in both orthogonal directions, ductility demands increase for all column types.

For both, interior and edge beams, the behavior of the ductility demands is similar for all values of α, as well as for the studied periods. In general, the higher ductility demands occur for lower period values and ductility demand decrease as the periods of the model increase. The critical case for which the highest ductility demands are reached corresponds to those models that have the same periods in both orthogonal directions. This beam behavior is applicable for both values of Q = 2 and Q = 4.

From the analysis of the data contained in figures 9.1 to 9.4, can be concluded that in order to achieve the closest average ductility demand for beams and columns to that used in the design of the models, it should be used in the design a value of $\alpha = 0.1$ for Q = 2 with a variation coefficient of ductility demands equal to 0.11, and $\alpha = 0.5$ for Q = 4 with a variation coefficient variation of ductility demands equal to 0.08.

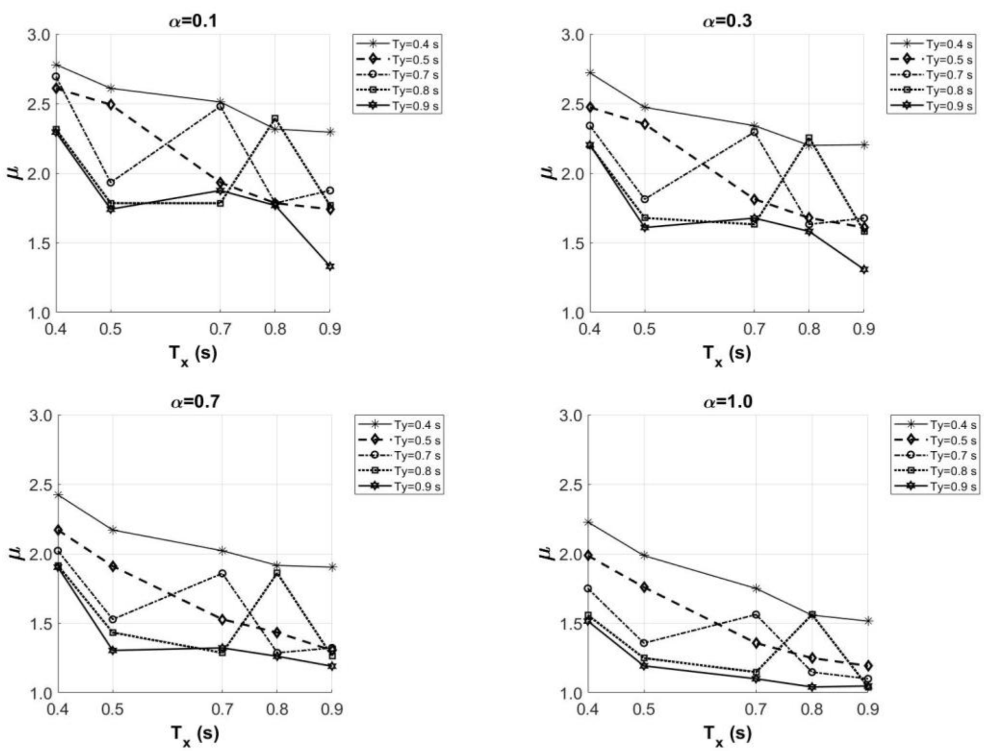


Figure 9.1. Ductility demand variation in corner columns for models designed with Q = 2 (firm soil earthquake records)

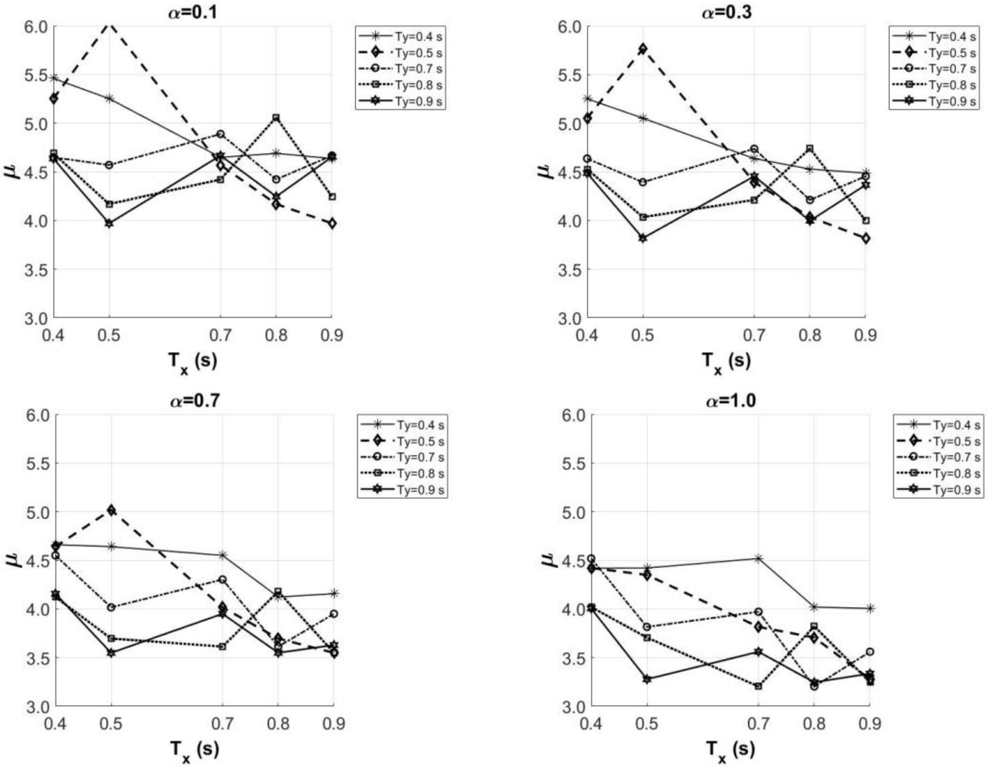


Figure 9.2. Ductility demand variation in corner columns for models designed with Q = 4 (firm soil earthquake records)

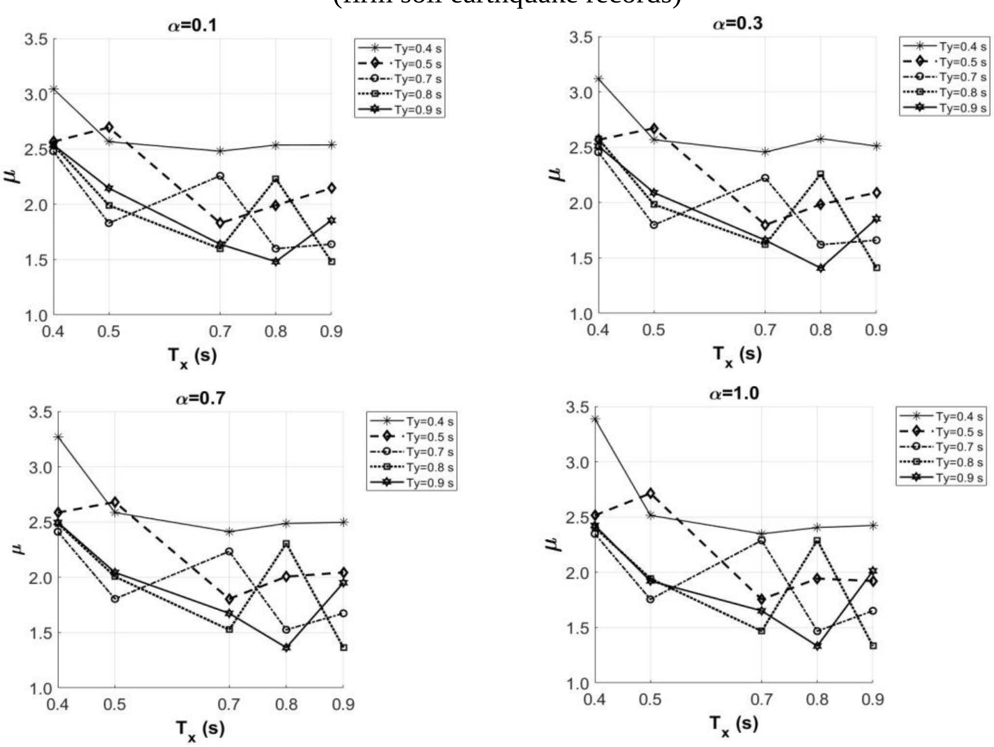


Figure 9.3. Ductility demand variation in edge beams for models designed with Q = 2 (firm soil earthquake records)

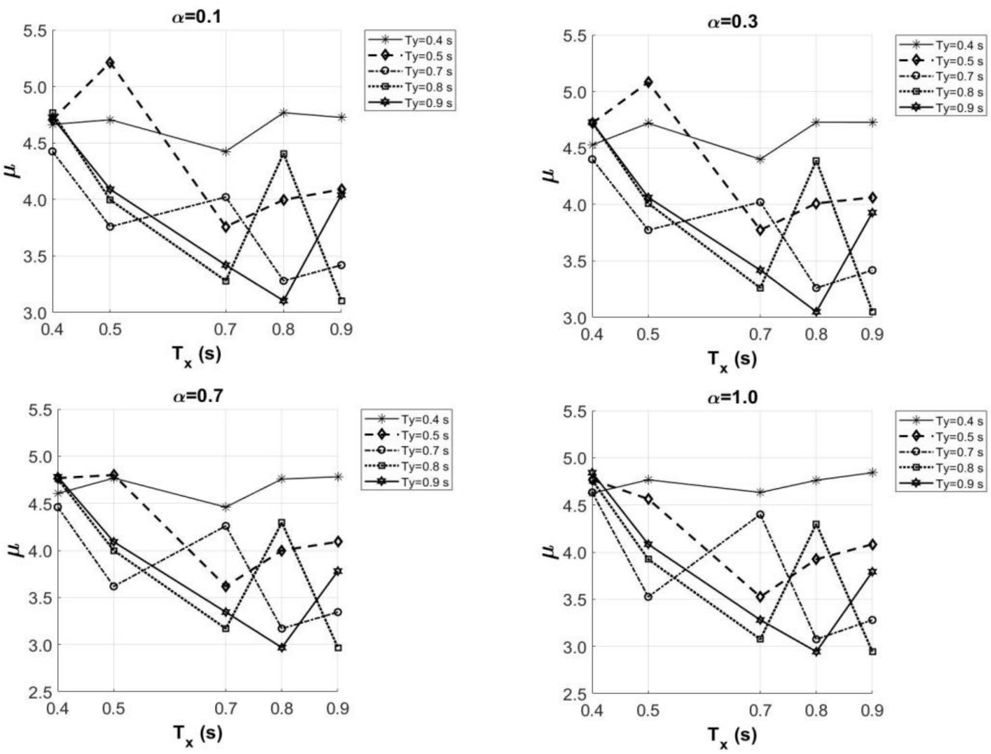


Figure 9.4. Ductility demand variation in edge beams for models designed with Q = 4 (firm soil earthquake records)

Figures 10.1, 10.2, 10.3 y 10.4 show the results of maximum ductility demands for soft-soil models considering all different earthquake records, incidence angles and α values. Figures 10.1 and 10.2 show results for corner columns and figs. 10.2 and 10.3 for edge beams. The horizontal axis contains the translation period of the models in the X direction, while the vertical axis shows the ductility demands. Different plots in each graph correspond to a particular translation period of the model in the Y direction. The behavior of the edge columns is similar to that of corner columns (figs. 10.1 and 10.2). In general the ductility demands of central columns are closer to the design ductility Q than those obtained for corner and edge columns.

In comparison with the firm-soil models, the variation in ductility demands in all types of columns is smaller for soft-soil models than for firm-soil models. This behavior is observed for all values of α and for all orthogonal periods. This indicates that soft-soil models have a greater stability in the variation of the ductility demands of columns, regardless of the translation orthogonal periods of the models. It was also observed that models with equal translation periods in both directions exhibit an increase in ductility demands. This behavior is observed for all column types.

For interior and edge beams, the behavior of ductility demands is similar for all analyzed α values. Such behavior occurs for both values of Q = 2 and Q = 4. As for the models of firm soil, the models whose periods of vibration are equal in both orthogonal directions show an increases in ductility demands. In the case of soft soil, it can be concluded that in order to reach ductility demands of beams and columns close to that supposed in the design, it should be used a value of α = 1.0 for Q = 2 with a variation coefficient of ductility demands equal to 0.09, and α = 0.7 for Q = 4 with a variation coefficient of ductility demands equal to 0.08.

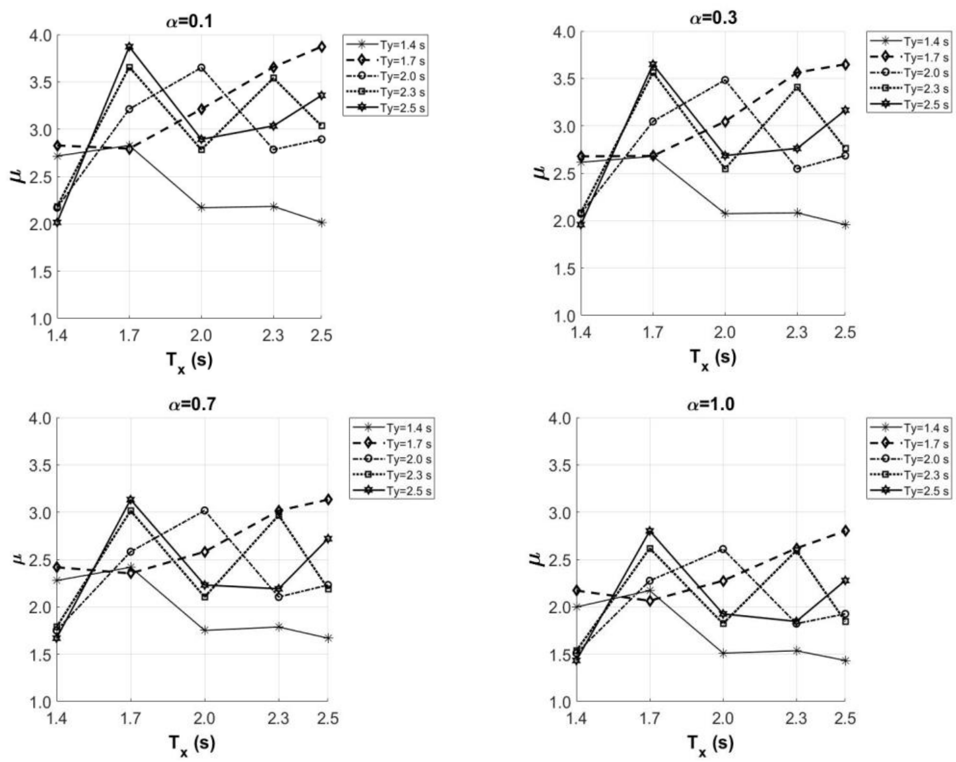


Figure 10.1 Ductility demand variation in corner columns for models designed with Q = 2 (soft soil earthquake records)

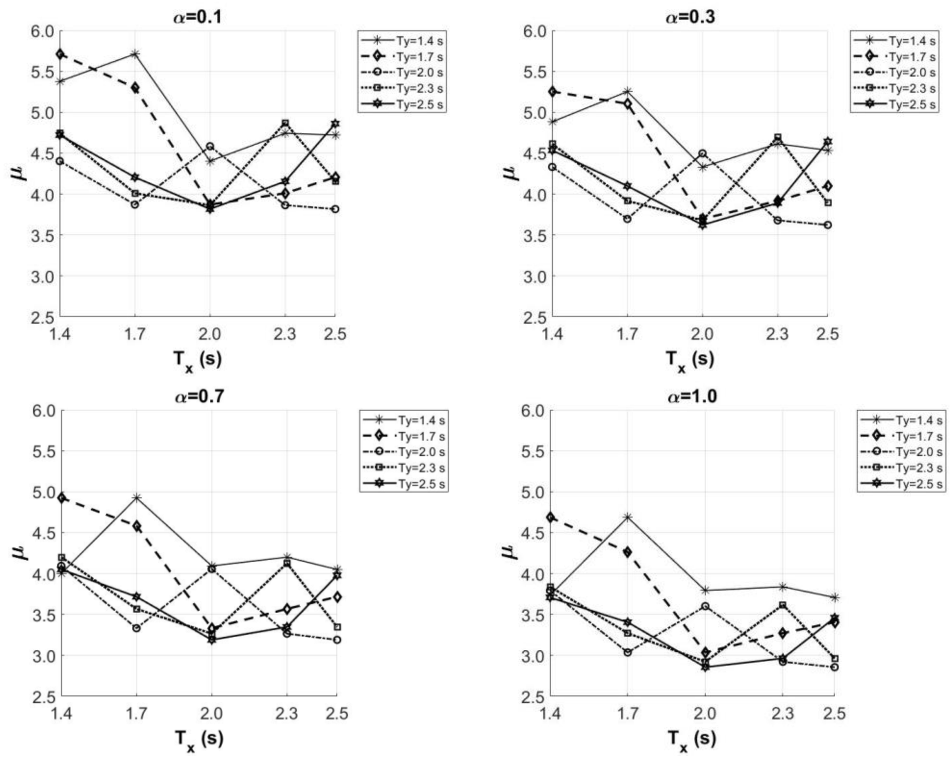


Figure 10.2 Ductility demand variation in corner columns for models designed with Q = 4 (soft soil earthquake records)

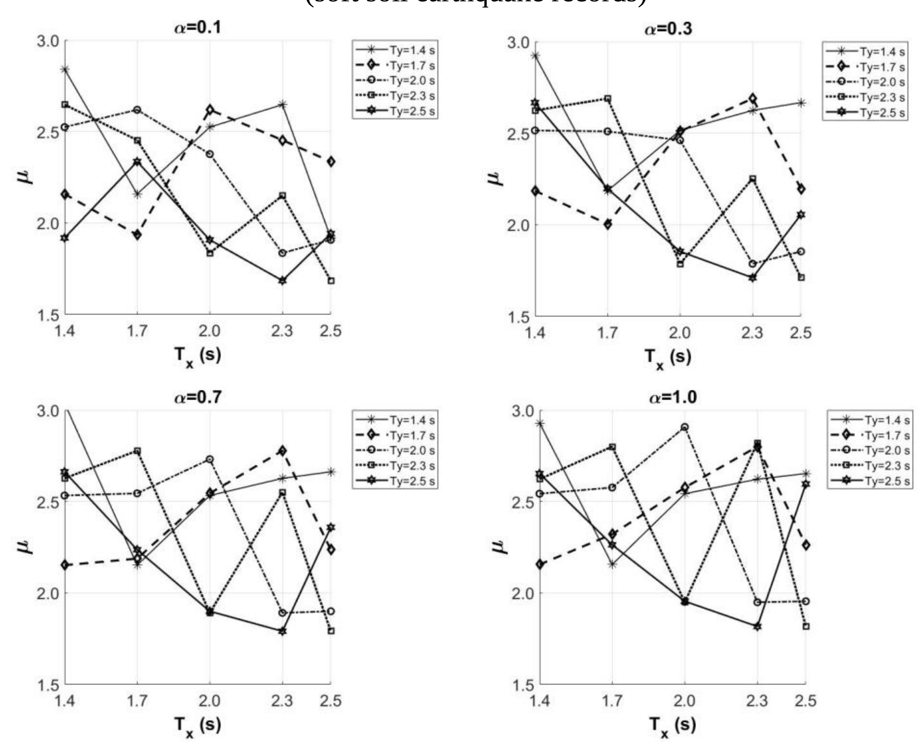


Figure 10.3 Ductility demand variation in edge beams for models designed with Q = 2 (soft soil earthquake records)

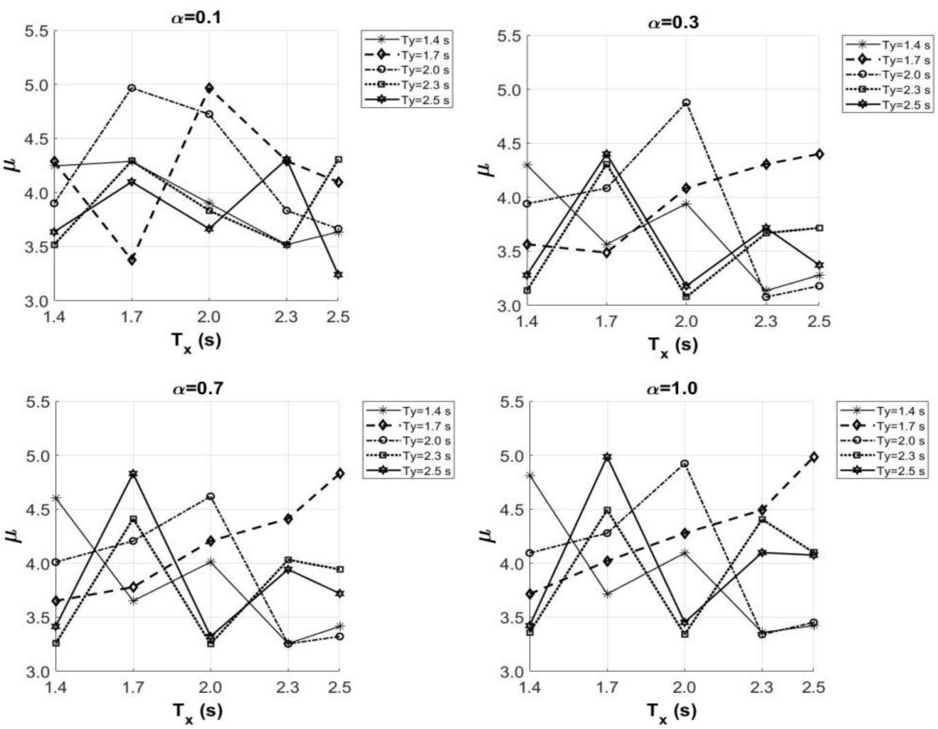


Figure 10.4 Ductility demand variation in edge beams for models designed with Q = 4 (soft soil earthquake records)

6.5 Total cost of all models

Figures 11.1, 11.2, 11.3 and 11.4 show the total cost of analyzed models including all possible values for the studied variables (earthquake records, incidence angles, α and Q values). The horizontal axis of the graphs contains the translation period of the models in the X direction, and the vertical axis contains the normalized total cost with respect to the initial cost of the building, which is called C_E*. Each graph corresponds to a particular value of the model translation period in the Y direction, and each plot in different graphs corresponds to a specific α value. The optimum α is defined as the one for which the lowest normalized total cost is reached. In general, it is observed that the optimum α varies in accordance with orthogonal translation periods of the models, soil conditions and design ductility. Tables 6 and 7 show the optimal value of α. On average, taking into account all possible values for the variables involved in the study, the optimum α value for firm soil is α = 0.85 for Q = 2.0 with a variation coefficient equal to 0.4 and α = 0.9 for Q = 4.0 with a variation coefficient equal to 0.15. For soft soil, the average optimum α values are α = 0.20 for Q = 2 with a variation coefficient equal to 1.3 and α = 0.3 for Q = 4 with a variation coefficient equal to 0.90.

For firm soil and Q = 2 the maximum and minimum normalized total costs (C_E*) are 2.1 and 1.7, respectively. These values represent a variation of C_E*close to 24%. For Q = 4.0, the maximum and minimum normalized total costs are 2.1 and 1.6 which represent a variation of 31%.

For soft soil, the maximum and minimum C_E* for Q = 2.0 are 2.5 and 1.9, respectively, while for Q = 4.0 they are 2.7 and 2, respectively. In this case, the variation between the maximum and minimum normalized costs are 31% for Q = 2.0 and 35% for Q = 4.0.

In general, the highest normalized total costs are reached in soft soil. The selection of a particular α value in the design of the models may represent a maximum variation in the total cost of the building of 31% for firm soil and 35% for soft soil.

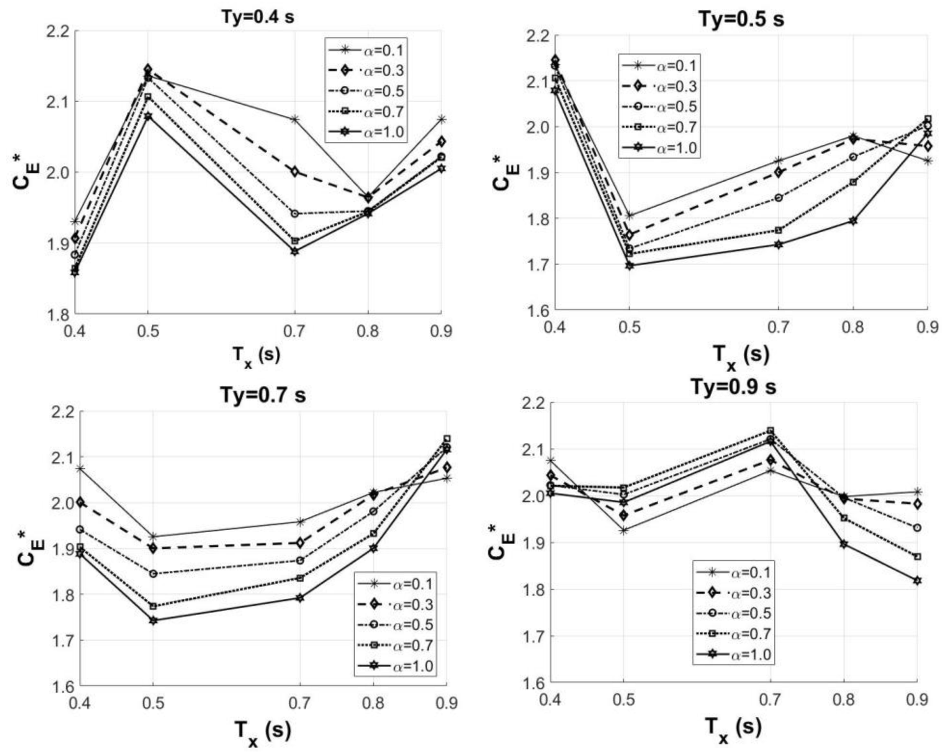


Figure 11.1 Normalized total cost (firm soil, Q = 2)

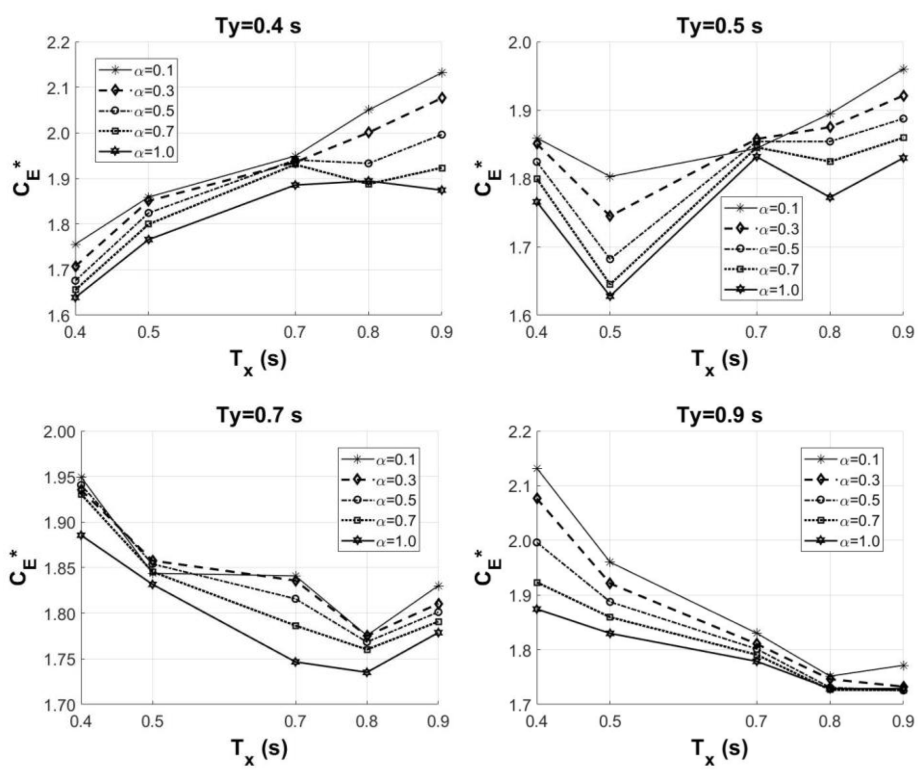


Figure 11.2 Normalized total cost (firm soil, Q = 4)

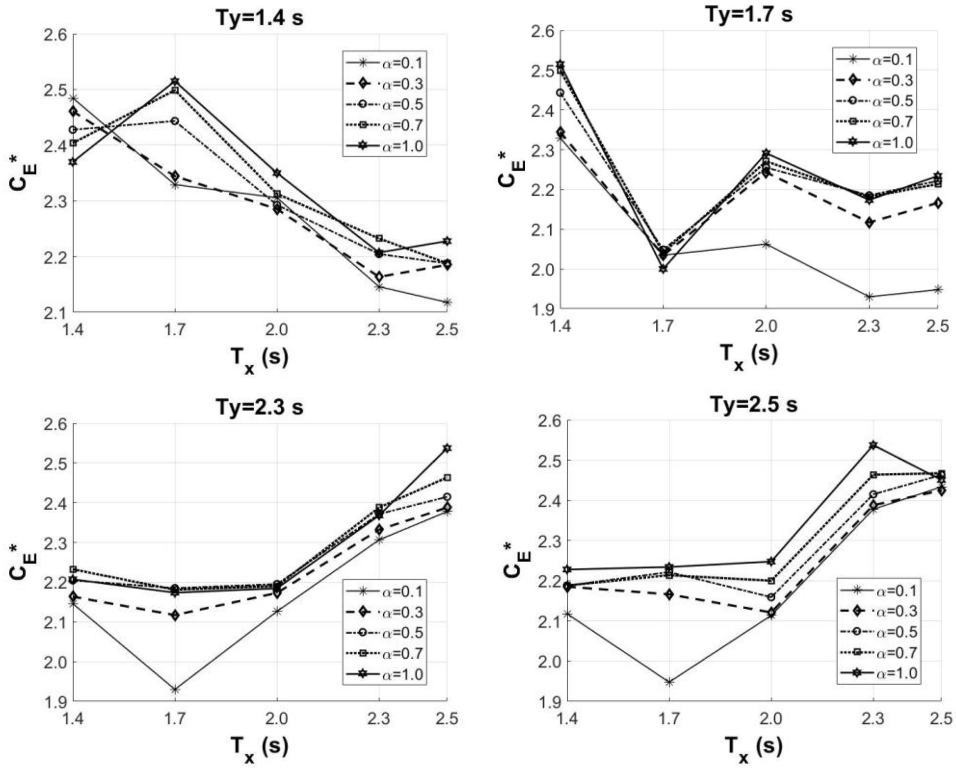


Figure 11.3 Normalized total cost (soft soil, Q = 2)

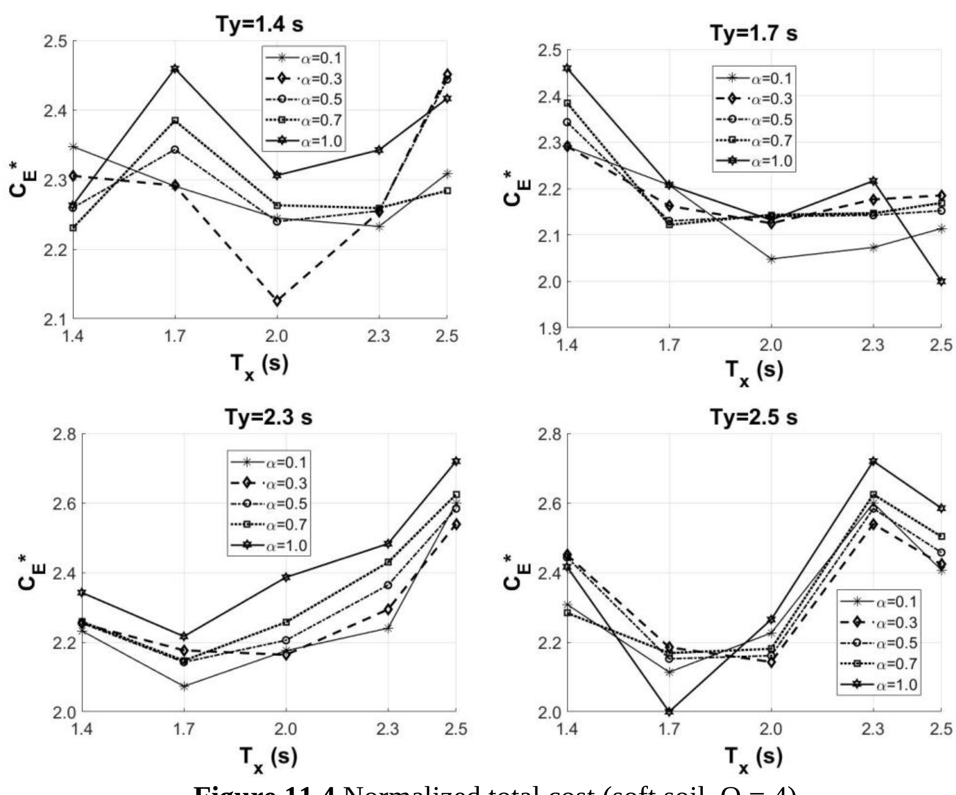


Figure 11.4 Normalized total cost (soft soil, Q = 4)

Table 7. Optimal values of α (firm soil)

	Q = 2					Q = 4				
Ty \ Tx	**0.4 s**	**0.5 s**	**0.7 s**	**0.8 s**	**0.9 s**	**0.4 s**	**0.5 s**	**0.7 s**	**0.8 s**	**0.9 s**
0.4 s	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.7	1.0
0.5 s		1.0	1.0	1.0	0.1		1.0	1.0	1.0	1.0
0.7 s			1.0	1.0	0.1			1.0	1.0	1.0
0.8 s	Symmetric			1.0	1.0	Symmetric			0.7	0.7
0.9 s					1.0					0.7

Table 8. Optimal values of α (soft soil)

	Q = 2					Q = 4				
Ty \ Tx	**1.4 s**	**1.7 s**	**2.0 s**	**2.3 s**	**2.5 s**	**1.4 s**	**1.7 s**	**2.0 s**	**2.3 s**	**2.5 s**
1.4 s	1.0	0.1	0.3	0.1	0.1	0.7	0.1	0.3	0.1	0.7
1.7 s		1.0	0.1	0.1	0.1		0.7	0.1	0.1	1.0
2.0 s			0.1	0.1	0.1			0.1	0.3	0.3
2.3 s	Symmetric			0.1	0.1	Symmetric			0.1	0.3
2.5 s					0.3					0.1

7. CONCLUSIONS

The inelastic behavior of different structural models with different translation periods in both orthogonal directions was studied. For each model, the incidence angle of the earthquake was varied from 0° to 90 ° with increments of 10°, as well as the combination of the horizontal orthogonal seismic effects used for design (α = 10%, 30%, 50%, 70% and 100%). 12 seismic records, 6 of firm soil and 6 of soft soil, were analyzed. Two values of design ductility demands were considered, Q = 2 and Q = 4. Based on these parameters, different analysis cases were carried out. For each one, the ductility demands of columns and beams of the models were evaluated, as well as for interstory drifts. With these interstory drifts the damage of the models was estimated for each case. The total cost of the models was evaluated, considering the building initial cost, repair costs, content losses, income loss costs, and losses due to injured and deceased people.

Based on the results of all the analyzed models and cases, the main conclusions are as follows.

Earthquake angle of incidence

Analyzing the variation between the maximum and minimum ductility demands of studied models as a function of the earthquake incidence angle and considering all α values. It was observed for Q = 2.0 that columns of soft-soil models has a greater difference between maximum and minimum ductility demands than the corresponding ones to firm soil. The difference for the soft-soil models columns is 15% greater than the difference for firm soil. For Q = 4, the behavior is reversed, the columns of the firm-soil models have a greater difference between the maximum and minimum ductility demands than that for columns of soft-soil models. The difference for the firm-soil models columns is 15% greater than that for soft soil. For beams, regardless Q values, the soft-soil models exhibit a greater difference between maximum and minimum ductility demands than the corresponding ones to firm-soil models. The difference for beams of soft-soil models is 12% bigger than the difference for beams of firm-soil models.

In general, it was observed that soft-soil models reach higher ductility demands in beams and columns than the firm-soil models. Also, the variation coefficients of the ductility demands for soft-soil models are higher than those corresponding to firm-soil models. For firm soil, the angle of incidence of the earthquake causes a difference between the calculated ductility demands and the assumed design ductility of up to 40%. In the case of soft soil, this difference is close to 60%.

Earthquake influence

Taking into account all different analyzed models for soft soil earthquake records, the difference between the maximum and minimum ductility demands in columns, considering all analyzed earthquakes, all incidence angles and all α values, is 40% for Q = 2 and 25% for Q = 4. For firm soil, this difference is 45%

for Q = 2 and 30% for Q = 4. In the case of beams, the difference is lower than 10% considering the two types of soil (firm and soft) and both Q values.

Ductility demands

For both soil conditions (firm and soft), and considering all possible values for all variables included in the study (model periods, earthquake, incidence angle, α and Q), it was observed that the differences between maximum and minimum ductility demands are lower for interior columns and edge beams than for another types of columns and beams. Moreover, it was observed that the differences corresponding to firm-soil models are lower in comparison with those corresponding ones to soft-soil models, regardless the orthogonal periods of the models.

It can be concluded that in order to achieve the closest average ductility demand for beams and columns to that used in the design, it should be used for firm soil a value of $\alpha = 0.1$ for Q = 2 with a variation coefficient of ductility demands equal to 0.11, and $\alpha = 0.5$ for Q = 4 with a variation coefficient of ductility demands equal to 0.08. In the case of soft soil, it should be used a value of $\alpha = 1.0$ for Q = 2 with a variation coefficient of ductility demands equal to 0.09, and $\alpha = 0.7$ for Q = 4 with a variation coefficient of ductility demands equal to 0.08.

In general, the ductility demands are greater for soft-soil models than for firm-soil models. It was also observed that models with equal orthogonal periods of vibration reach the highest ductility demands, regardless soil conditions.

Total cost

In general, it is observed that the optimum α value which minimizes the total cost of analyzed building models varies in accordance with orthogonal translation periods of the models, soil conditions and design ductility

For firm soil, the average of the optimum α values for all analyzed cases is $\bar{\alpha}$ = 0.85 for Q = 2 with a variation coefficient equal to 0.4 and $\bar{\alpha}$ = 0.9 for Q = 4 with a variation coefficient equal to 0.15. For soft soil, $\bar{\alpha}$ = 0.20 for Q = 2 with a variation coefficient equal to 1.3 and $\bar{\alpha}$ = 0.30 for Q = 4 with a variation coefficient equal to 0.9.

In general, the total cost of the buildings on firm soil can reach a ratio up to 2 in relation with their initial cost, depending on the selected α value. For soft soil, this ratio is close to 2.7. The value of α is relevant in the determination of the total cost of buildings since for the firm-soil models the variation between the maximum and minimum total cost is 31%, while for the soft-soil models is 35%.

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