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# INELASTIC SEISMIC RESPONSE OF MULTISTOREY R/C BUILDINGS DESIGNED ON THE BASIS OF LINEAR TIME HISTORY ANALYSIS

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## *Abstract*

Modern seismic codes suggest, among others, the use of Linear Response History Analysis (LRHA) for the design of buildings. According to LRHA, a three-dimensional mathematical model is analysed using simultaneously imposed consistent pairs of accelerograms along the structural axes. In general, the seismic design of R/C structural members is controlled by the simultaneous action of three response parameters. For example, a column in a 3D frame should be proportioned to resist axial force and two bending moments. The value of any response parameter computed by LRHA is a function of time. The three response parameters acting in a column's section do not attain their maximum values at the same time instant. Besides, many studies have shown that the response depends strongly on the seismic incident angle.

The issue of the appropriate selection of the internal forces needed for the calculation of the longitudinal reinforcement in Reinforced Concrete (R/C) structural elements within the context of LRHA was investigated in the past and a study has been published in which four different procedures are presented for the appropriate selection of sectional forces. According to the three procedures, the accelerograms are applied along the structural axes, whereas in the fourth procedure, the orientation of the ground-motion that produces the maximum normal stress in the cross section under examination is considered. Then the corresponding internal forces (axial force and two bending moments) are used for the determination of the longitudinal reinforcement. The parametric analysis in single story R/C buildings revealed that the required reinforcement is strongly affected by the procedure used to select the design sectional forces in the frame elements. Furthermore, the preliminary studyof the effectiveness of the four aforementioned procedures in the inelastic structural response of a single-storey R/C asymmetric building showed that the procedure which is based on the maximum normal stress over all seismic incident angles is more efficient for the design of R/C frame elements.

The objective of the present work is to evaluate the four procedures for selecting the sectional forces for design purposes in the nonlinear range of behavior in case of multistorey buildings. For this purpose amultistorey asymmetric building designed using thefour procedures is analyzed by Nonlinear Response History Analysis (NRHA) underseven bi-directional earthquake ground motions. The two horizontal accelerograms of each ground motion are applied along horizontal orthogonal axes forming an angle θ=0°, 30°, 60°, …, …, 330° with the structural axes. Moreover, two different seismic intensity levels are considered by using two appropriate multipliers for each ground motion. For the evaluation of inelastic structural behavior the maximum interstorey drift ratio of the building is computed. The analyses results show that the overall damage state of the building is significantly affected by the procedure used to select the design sectional forces. The building designed using the procedure based on the maximum normal stress over all seismic incident angles shows the best seismic performance.

*Keywords:seismic design; R/C buildings; time history analysis; internal forces selection; inelastic response*

## 1. Introduction

Current seismic code provisions [1-6] state that one of the methods which can be used for the seismic analysis and design of R/C structures is the Linear Response History Analysis (LRHA). Moreover, manyengineers have already been using this method for analyses in advanced applications, such as bridges, dams, nuclear facilities etc. [7-9]. In this method a three-dimensional mathematical model is analysed using simultaneously imposed consistent pairs of earthquake ground motion records along each of the two horizontal structural axes (with a few exceptions, the vertical component of the ground motion is allowed to be ignored as its influence on seismic response is considered negligible). Alternatively, the structure is analyzed separately due to each horizontal component applied along each structural axis and then the action effects are combined according to the percentage (30%) combination rule. The application of LRHA induces manyquestions regarding, among others, the representative collection and correct scaling of ground motions, the choice of the excitation's incident angle, and the proper (i.e. safe but not too conservative) selection of the frame's sectional forces required for the final design of the R/C frame elements. A review of code provisions regarding the aforementioned aspects reveals that they are lacking the necessary definiteness. Particularly important issues are the right choice of the incident angle and the proper selection of the frame's sectional forces, because both of them strongly affect the response quantities and, consequently, the reinforcement steel ratio.

Concerning the angle of seismic incidence, FEMA 356 [2] and ASCE 41-06 [3] state that the axes of the ground motion "shall, in general, be aligned with the principal axes of the structure". According to EC8 [1] the seismic action shall "be applied along all relevant horizontal directions in both positive and negative polarity." However, no specifications are made regarding the relevant horizontal directions with the exception of buildings with resisting elements in two perpendicular directions in which these two directions shall be considered as the relevant ones. The lack of specific provisions concerning the axes of the seismic input leads to the common engineering pracice of applying the horizontal seismic components along the structural axes, which leads to significant underestimation of seismic demands not only in the linear, but also in the nonlinear range of behavior [10-15]. Regarding the combination of sectional forces which should be used for design purposes, none of the seismic codes defines which is the proper (i.e., safe but not too conservative) combination. Most seismic code provisions specify that when three time history data sets are used as seismic input, the maximum value of each response parameter must be used for design, while in case of seven or more time history data sets the average value of each response parameter may be permitted to determine design acceptability.

The issue of the appropriate selection of the internal forces needed for the calculation of the reinforcement in R/C structural elements within the context of LRHA was investigated by Kostinakis et al. [16]. They presented four different procedures in an attempt to realistically interpret pertinent code provisions. In order to compare the four procedures an extensive parametric study was conducted using single-story R/C buildings. Furthermore, in a preliminary study, the same authors [17] evaluated the effectiveness of the four aforementioned procedures in the inelastic structural response of a single-storey R/C building.

The present paper aims to contribute to the development of design solutions towards a performance-based design framework using LTHA. To accomplish this aim the ability of the four abovementioned procedures in the nonlinear range of behavior is further investigated. An asymmetric 3-storey building designed using the four procedures for selecting the sectional forces in order to dermine the reinforcing steel ratio is analyzed by Nonlinear Response History Analysis (NRHA) under seven bi-directional earthquake ground motions. The two horizontal accelerograms of each ground motion are applied along horizontal orthogonal axes forming an angle θ=0°, 30°, 60°, …, …, 330° with the structural axes. Moreover, two different seismic intensity levels are considered by using appropriate multipliers fo reach ground motion. For the evaluation of inelastic structural behavior global damage measures, such as the máximum and average interstoey drift ratio and the máximum floor acceleration, are computed. The analyses results show that the overall damage state of the building is significantly affected by the procedure used to select the design sectional forces.

## 2. Design procedures for selection of the sectional forces within the context of LRHA

### 2.1 Critical orientation and maximum response

Athanatopoulou [10] has developed analytical formulae for the determination of the critical angle of seismic incidence and the corresponding maximum value of any response quantity in structures subjected to two horizontal seismic components within the context of LRHA. The structure is subjected to bidirectional horizontal seismic motion consisting of the accelerograms $\ddot{u}_{ag}(t)$and $\ddot{u}_{bg}(t)$. As the direction of the seismic motion is unknown, they can form any angle θ with the x and y structural axes (Fig. 1(a)). We consider two orientations of the seismic excitation:

(i) Excitation 'α0': The accelerograms $\ddot{u}_{ag}(t)$ and $\ddot{u}_{bg}(t)$ are applied along the axes x and y, respectively, i.e. the angle of seismic incidence is θ=0º (Fig. 1(b)). A typical response quantity R is denoted as $R,_{\alpha 0}$.

(ii) Excitation 'α90': The accelerograms $\ddot{u}_{ag}(t)$ and $\ddot{u}_{bg}(t)$ are applied along the axes y and x, respectively, i.e. the angle of seismic incidence is θ=90º (Fig. 1(c)). A typical response quantity R is denoted as $R,_{\alpha 90}$.

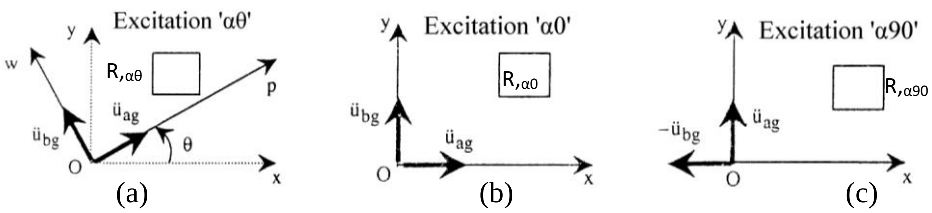


Fig. 1 - Excitations 'αθ' (a), 'α0' (b) and 'α90' (c)

It has been proved [10] that the maximum value of a response parameter for any angle θo f seismic incidence is given, as a function of time, by the relation:

$$R_0(t) = [R,^2_{\alpha 0}(t) + R,^2_{\alpha 90}(t)]^{1/2} \tag{1}$$

The plot of the function $\pm R_0(t)$ provides the maximum/minimum value of the required response parameter as well as the time instant $t_{cr}$ at which the maximum/minimum occurs.

$$\max R = +R_0(t_{cr}) \quad \text{and} \quad \min R = -R_0(t_{cr}) \tag{2}$$

The corresponding critical angles $\theta_{cr1}$ (maximum value) and $\theta_{cr2}$ (minimum value) are given by the relations:

$$\theta_{cr1} = \tan^{-1}\left(\frac{R,_{\alpha 90}(t_{cr})}{R,_{\alpha 0}(t_{cr})}\right) \quad \text{and} \quad \theta_{cr2} = \theta_{cr1}\text{-}\pi \tag{3}$$

The value of any other response parameter R′ at the time instant $t_{cr}$ for incident angle $\theta_{cri}$ (i=1, 2) is computed by the relation:

$$R'(\theta_{cri}, t_{cr}) = R',_{\alpha 0}(t_{cr}) \cdot \cos\theta_{cri} + R',_{\alpha 90}(t_{cr}) \cdot \sin\theta_{cri} \tag{4}$$

In the following four different procedures for the selection of sectional forces which will be used for the calculation of the reinforcement steel ratio are briefly presented [16].

### 2.2 Procedure of extreme stresses for angle θ=0º ($MS_{ex}0$)

According to this method the time histories of the normal stresses $\sigma_A(t)_{,\alpha0}$, $\sigma_B(t)_{,\alpha0}$, $\sigma_C(t)_{,\alpha0}$, $\sigma_D(t)_{,\alpha0}$ at the four corners A, B, C and D of a rectangular cross section are computed. Then, the maximum and minimum values of the stresses, as well as the corresponding time instants $t_1$ and $t_2$ are determined. The sectional forces $N(t_i)_{,\alpha0}$, $M_\xi(t_i)_{,\alpha0}$ and $M_\eta(t_i)_{,\alpha0}$ (i=1,2), which correspond to maximum and minimum values of the normal stresses, are considered as the design combinations. Hence, at the four corners of any relevant rectangular cross section the following eight combinations have to be considered (Table 1).

Table 1 - Design combinations for method $MS_{ex}0$

| | | | |
|---|---|---|---|
| $\max\sigma_{A,\alpha0}$ | $N_{,\max\sigma A,\alpha0}$ | $M_{\xi,\max\sigma A,\alpha0}$ | $M_{\eta,\max\sigma A,\alpha0}$ |
| $\min\sigma_{A,\alpha0}$ | $N_{,\min\sigma A,\alpha0}$ | $M_{\xi,\min\sigma A,\alpha0}$ | $M_{\eta,\min\sigma A,\alpha0}$ |
| $\max\sigma_{B,\alpha0}$ | $N_{,\max\sigma B,\alpha0}$ | $M_{\xi,\max\sigma B,\alpha0}$ | $M_{\eta,\max\sigma B,\alpha0}$ |
| $\min\sigma_{B,\alpha0}$ | $N_{,\min\sigma B,\alpha0}$ | $M_{\xi,\min\sigma B,\alpha0}$ | $M_{\eta,\min\sigma B,\alpha0}$ |
| $\max\sigma_{C,\alpha0}$ | $N_{,\max\sigma C,\alpha0}$ | $M_{\xi,\max\sigma C,\alpha0}$ | $M_{\eta,\max\sigma C,\alpha0}$ |
| $\min\sigma_{C,\alpha0}$ | $N_{,\min\sigma C,\alpha0}$ | $M_{\xi,\min\sigma C,\alpha0}$ | $M_{\eta,\min\sigma C,\alpha0}$ |
| $\max\sigma_{D,\alpha0}$ | $N_{,\max\sigma D,\alpha0}$ | $M_{\xi,\max\sigma D,\alpha0}$ | $M_{\eta,\max\sigma D,\alpha0}$ |
| $\min\sigma_{D,\alpha0}$ | $N_{,\min\sigma D,\alpha0}$ | $M_{\xi,\min\sigma D,\alpha0}$ | $M_{\eta,\min\sigma D,\alpha0}$ |

## 2.3 Procedure of maximum absolute forces for angle θ=0° ($MF_{abs}0$)

According to this method the maximum absolute values of the response parameters $N(t)_{,\alpha0}$, $M_\xi(t)_{,\alpha0}$ and $M_\eta(t)_{,\alpha0}$ are used for design purposes. The design combinations for any relevant cross section are presented in Table 2.

Table 2 - Design combinations for method $MF_{abs}0$

| | | |
|---|---|---|
| $\max\lvert N_{,\alpha0}\rvert$ | $\max\lvert M_{\xi,\alpha0}\rvert$ | $\max\lvert M_{\eta,\alpha0}\rvert$ |
| $\max\lvert N_{,\alpha0}\rvert$ | $\max\lvert M_{\xi,\alpha0}\rvert$ | $-\max\lvert M_{\eta,\alpha0}\rvert$ |
| $\max\lvert N_{,\alpha0}\rvert$ | $-\max\lvert M_{\xi,\alpha0}\rvert$ | $\max\lvert M_{\eta,\alpha0}\rvert$ |
| $\max\lvert N_{,\alpha0}\rvert$ | $-\max\lvert M_{\xi,\alpha0}\rvert$ | $-\max\lvert M_{\eta,\alpha0}\rvert$ |
| $-\max\lvert N_{,\alpha0}\rvert$ | $\max\lvert M_{\xi,\alpha0}\rvert$ | $\max\lvert M_{\eta,\alpha0}\rvert$ |
| $-\max\lvert N_{,\alpha0}\rvert$ | $\max\lvert M_{\xi,\alpha0}\rvert$ | $-\max\lvert M_{\eta,\alpha0}\rvert$ |
| $-\max\lvert N_{,\alpha0}\rvert$ | $-\max\lvert M_{\xi,\alpha0}\rvert$ | $\max\lvert M_{\eta,\alpha0}\rvert$ |
| $-\max\lvert N_{,\alpha0}\rvert$ | $-\max\lvert M_{\xi,\alpha0}\rvert$ | $-\max\lvert M_{\eta,\alpha0}\rvert$ |

## 2.4 Procedure of 30% rule (M30)

According to this method two response history analyses, for uni-directional inputs $\ddot{u}_{ag}(t)$ and $\ddot{u}_{bg}(t)$ along the structural axes x and y, respectively are performed.The time histories of the response quantities $N(t)_{,xa}$, $M_\xi(t)_{,xa}$ and $M_\eta(t)_{,xa}$, as well as $N(t)_{,yb}$, $M_\xi(t)_{,yb}$, $M_\eta(t)_{,yb}$ at any relevant cross section are computed and their maximum absolute values are determined. Then the 30% directional combination rule is applied. The design combinations for any relevant cross section are presented in Table 3.

Table 3 - Design combinations for method M30

| | | |
|---|---|---|
| $\max\lvert N_{,xa}\rvert+0.3\max\lvert N_{,yb}\rvert$ | $\max\lvert M_{\xi,xa}\rvert+0.3\max\lvert M_{\xi,yb}\rvert$ | $\max\lvert M_{\eta,xa}\rvert+0.3\max\lvert M_{\eta,yb}\rvert$ |
| $\max\lvert N_{,xa}\rvert-0.3\max\lvert N_{,yb}\rvert$ | $\max\lvert M_{\xi,xa}\rvert-0.3\max\lvert M_{\xi,yb}\rvert$ | $\max\lvert M_{\eta,xa}\rvert-0.3\max\lvert M_{\eta,yb}\rvert$ |
| $-\max\lvert N_{,xa}\rvert+0.3\max\lvert N_{,yb}\rvert$ | $-\max\lvert M_{\xi,xa}\rvert+0.3\max\lvert M_{\xi,yb}\rvert$ | $-\max\lvert M_{\eta,xa}\rvert+0.3\max\lvert M_{\eta,yb}\rvert$ |
| $-\max\lvert N_{,xa}\rvert-0.3\max\lvert N_{,yb}\rvert$ | $-\max\lvert M_{\xi,xa}\rvert-0.3\max\lvert M_{\xi,yb}\rvert$ | $-\max\lvert M_{\eta,xa}\rvert-0.3\max\lvert M_{\eta,yb}\rvert$ |
| $0.3\max\lvert N_{,xa}\rvert+\max\lvert N_{,yb}\rvert$ | $0.3\max\lvert M_{\xi,xa}\rvert+\max\lvert M_{\xi,yb}\rvert$ | $0.3\max\lvert M_{\eta,xa}\rvert+\max\lvert M_{\eta,yb}\rvert$ |
| $0.3\max\lvert N_{,xa}\rvert-\max\lvert N_{,yb}\rvert$ | $0.3\max\lvert M_{\xi,xa}\rvert-\max\lvert M_{\xi,yb}\rvert$ | $0.3\max\lvert M_{\eta,xa}\rvert-\max\lvert M_{\eta,yb}\rvert$ |



 

| | | |
|---|---|---|
| $-0.3\max\|N_{,xa}\|+\max\|N_{,yb}\|$ | $-0.3\max\|M_{\xi,xa}\|+\max\|M_{\xi,yb}\|$ | $-0.3\max\|M_{\eta,xa}\|+\max\|M_{\eta,yb}\|$ |
| $-0.3\max\|N_{,xa}\|-\max\|N_{,yb}\|$ | $-0.3\max\|M_{\xi,xa}\|-\max\|M_{\xi,yb}\|$ | $-0.3\max\|M_{\eta,xa}\|-\max\|M_{\eta,yb}\|$ |

2.5 Procedure of extreme stresses ($MS_{ex}$)

According to this method two response history analyses, under bi-directional excitation for incident angles $\alpha$=0° (Fig. 1(b)) and $\alpha$=90° (Fig. 1(c)), are performed.The time histories of the response quantities $N(t)_{,\alpha 0}$, $M_\xi(t)_{,\alpha 0}$ and $M_\eta(t)_{,\alpha 0}$, as well as of $N(t)_{,\alpha 90}$, $M_\xi(t)_{,\alpha 90}$, $M_\eta(t)_{,\alpha 90}$ at any relevant cross section are computed. Then, the time histories of the normal stresses ($\sigma_A(t)_{,\alpha 0}$, $\sigma_B(t)_{,\alpha 0}$, $\sigma_C(t)_{,\alpha 0}$, $\sigma_D(t)_{,\alpha 0}$ and $\sigma_A(t)_{,\alpha 90}$, $\sigma_B(t)_{,\alpha 90}$, $\sigma_C(t)_{,\alpha 90}$, $\sigma_D(t)_{,\alpha 90}$) at the four corners A, B, C and D of a rectangular cross section are calculated. Finally, using Eqns. (1-4), the maximum and minimum values of the stresses, the associated critical incident angles $\theta_{cr1}$ and $\theta_{cr2}$, as well as the time instant $t_{cr}$ are determined. The sectional forces corresponding to these maximum and minimum values of normal stresses are used for design purposes. The design combinations for any relevant rectangular cross section are presented in Table 4.

Table 4 - Design combinations for method $MS_{ex}$

| | | | |
|---|---|---|---|
| $\max\sigma_A$ | $N,_{\max\sigma A}$ | $M_\xi,_{\max\sigma A}$ | $M_\eta,_{\max\sigma A}$ |
| $\min\sigma_A$ | $N,_{\min\sigma A}$ | $M_\xi,_{\min\sigma A}$ | $M_\eta,_{\min\sigma A}$ |
| $\max\sigma_B$ | $N,_{\max\sigma B}$ | $M_\xi,_{\max\sigma B}$ | $M_\eta,_{\max\sigma B}$ |
| $\min\sigma_B$ | $N,_{\min\sigma B}$ | $M_\xi,_{\min\sigma B}$ | $M_\eta,_{\min\sigma B}$ |
| $\max\sigma_C$ | $N,_{\max\sigma C}$ | $M_\xi,_{\max\sigma C}$ | $M_\eta,_{\max\sigma C}$ |
| $\min\sigma_C$ | $N,_{\min\sigma C}$ | $M_\xi,_{\min\sigma C}$ | $M_\eta,_{\min\sigma C}$ |
| $\max\sigma_D$ | $N,_{\max\sigma D}$ | $M_\xi,_{\max\sigma D}$ | $M_\eta,_{\max\sigma D}$ |
| $\min\sigma_D$ | $N,_{\min\sigma D}$ | $M_\xi,_{\min\sigma D}$ | $M_\eta,_{\min\sigma D}$ |

## 3. Application - Methodology

3.1 Design of the building

A 3-storeyasymmetric R/C buildingwasdesigned using the four procedures presented in the previous section(Fig. 2).The distance between the mass centre and the stiffness centre, which defines the structural eccentricity $e_0$, fulfils the condition given in par. 4.2.3.2 of EC8 [1] and, therefore, the building displays a high degree of asymmetry and can be classified as irregular in plan.The design data (geometric and material properties) of the building are given in Table 5. For the building's modeling all basic recommendations of EC8 [1], such as the diaphragmatic behavior of the slabs, the rigid zones in the joint regions of beams/columns and beams/walls and the values of flexural and shear stiffness corresponding to cracked R/C elements were taken into consideration.



 

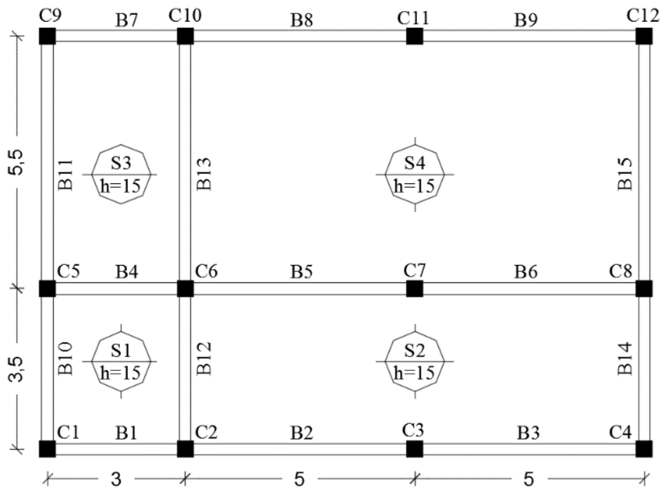


Fig. 2 –Plan view of the 3-storey asymmetric building

The building was subjected to a set of seven pairs of horizontal ground motion records (Table 6), as specify the most of the seismic code provisions (e.g. [1-3]). The seismic records were obtained from the PEER strong motion database [18]. For each strong motion pair LRHA were conducted. Ground motions were recorded on site class C of EC8 [1]during seismic events with magnitudes ($M_s$) between 5.7 and 7.4. The accelerograms were scaled so as to match the desgn spectrum of EC8 [1]. For each ground motion the longitudinal reinforcement steel ratios at every cross section of the building were calculated using the four methods described in section 2 and taking into account the design vertical loads.

Table 5 -Design data for the building.

| **Storeys' heights $H_i$** | **Behavior factor (q)** | **Concrete** | **Steel** | **Slab loads** | **Masonry loads** | **Design spectrum (EC8)** |
|---|---|---|---|---|---|---|
| 3.2m | 3.45 | *C20/25*<br>$E_c$=3•10$^7$kN/m$^2$<br>ν=0.2<br>w=25kN/m$^3$ | *S500B*<br>$E_s$=2•10$^8$kN/m$^2$<br>ν=0.3<br>w=78.5kN/m$^3$ | *Dead*:<br>G=1.0kN/m$^2$<br>*Live*:<br>Q=2.0kN/m$^2$ | *Perimetric beams*:<br>3.6kN/m$^2$<br>*Internal beams*:<br>2.1kN/m$^2$ | *Reference PGA*:<br>$a_{gR}$=0.24g<br>*Importance class*: II →$\gamma_I$=1<br>*Ground type*: C |

Table 6 - Ground motion records

| Date | Earthquake Name | Station Name | Magnitude ($M_s$) | Closest Distance (km) |
|---|---|---|---|---|
| 28/06/1992 | Landers | Yermo Fire Station | 7.4 | 24.9 |
| 18/10/1989 | Loma Prieta | Oakland - Title & Trust | 7.1 | 77.4 |
| 17/01/1994 | Northridge | Manhattan Beach - Manhattan | 6.7 | 42.0 |
| 24/11/1987 | Superstitn Hills | Calipatria Fire Station | 6.6 | 28.3 |
| 01/10/1987 | Whittier Narrows | Downey - Birchdale | 5.7 | 56.8 |
| 01/10/1987 | Whittier Narrows | Studio City - Coldwater Can | 5.7 | 28.7 |
| 12/11/1999 | Duzce, Turkey | Bolu | 7.3 | 17.6 |

3.2 Assessment of the building's seismic damage



 

For the modeling of the building's nonlinear behavior, plastic hinges located at the column and beam ends as well as at the base of the walls were used. The material inelasticity of the structural members was modeled by means of the seismic provisions of ASCE 41-13 [19]. It is important to notice that the effects of axial load-biaxial bending moment ($P$-$M_1$-$M_2$) interaction at column and wall hinges are taken into consideration by means of the interaction diagram implemented in the software used to conduct the analyses [20]. The yield moments as well as the parameters needed to determine the $P$-$M_1$-$M_2$ interaction diagram of the vertical elements' cross sections were determined by the same software [20]. More specifically, the plastic moments as well as the parameters needed to determine the interaction diagram of the column cross sections were calculated according to the reinforcement produced by each one of the four procedures. Note that the average value of the required longitudinal reinforcement computed by the application of the seven earthquake records was considered, according to the provisions of the most seismic codes. Therefore, four structural models were produced, each one corresponding to a different procedure to select the sets of sectional forces (different reinforcement). Then, the four models were analyzed by Nonlinear Response History Analysis (NRHA) for the seven earthquake ground motionstaking into account the design vertical loads.

In order to investigate the influence of the seismic intensity on the inelastic structural response of the building two different earthquake intensity levels are considered for the NRHA:a) Performance level of Significant Damage (SD) -according to EC8 (par. 2.1) which corresponds to a reference return period of 475 years and b) Performance level of Near Collapse (NC) -according to EC8 (par. 2.1) which corresponds to a reference return period of 2475 years. It must be noted that the building is designed for intensity level SD. To accomplish the two different earthquake intensity levels, each ground motion was multiplied by an appropriate scale factor.Furthermore, as the seismic incident angle with regard to structural axes is unknown, the two horizontal accelerograms of each ground motion were applied along horizontal orthogonal axes forming with the structural axes an angle $\theta$=0°, 30°, 60°, …, …, 330°. Thus for each pair of accelerograms and each intensity level 12 orientations were considered. As a consequence a total of 672 NRHA (7 earthquake records x 4 design procedures x 2 seismic intensity levels x 12 incident angles) were conducted in the present study.

For each one of these analyses, the damage stateof the building was determined. The seismic performance was expressed in the form of global response measures, namely, the Average Interstorey Drift Ratio (AIDR), the Maximum Interstorey Drift Ratio (MIDR) and the Maximum Floor Acceleration (MFA). The AIDR was calculated as the ratio of the maximum top displacement to the total height of the structure. It is the simplest of the response measures, suitable only for a coarse estimation of the seismic response of the frames. The interstorey drift ratio, calculated at every storey as the ratio of the maximum interstorey drift to the storey height, is a commonly used damage measure, which represents the deformation demand at the storey level. The MIDR represents the maximum storey-level deformation demand on the building and it is generally considered an effective indicator of the global structural and nonstructural damage of R/C buildings (e.g. [21-23]). The same parameter also characterises the mechanical, electrical and plumbing damage and also the damage of furniture, equipmentand other contents [21]. On the other hand, the response factor which has been associated with furniture and equipment damage is the maximum floor acceleration [21].

## 4. Analyses results

In order to assess the general trends exhibited by the aforementioned four procedures, the average value of the three damage measures considered herein for all the seven seismic records was computed. Figs. 3-8 illustrate the average values of AIDR, MIDR and MFA with the incident angle. In particular, the graphs of the damage indices vs incident angle were plotted separately for each performance level (Figs. 3-5 for SD and Figs. 6-8 for NC).



 

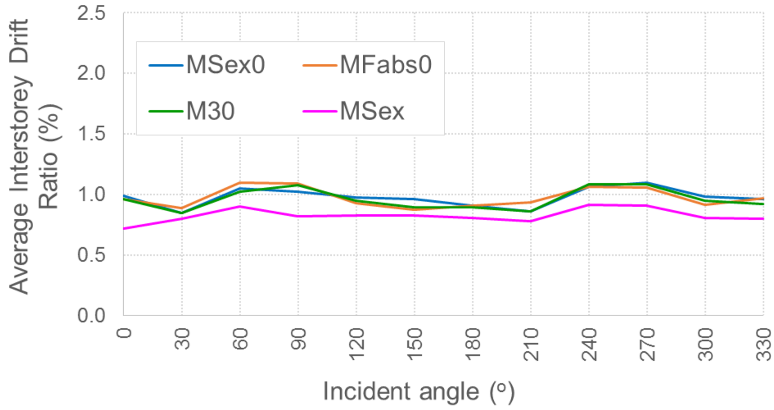


Fig. 3- Variation of AIDR with incident angle for performance level SD

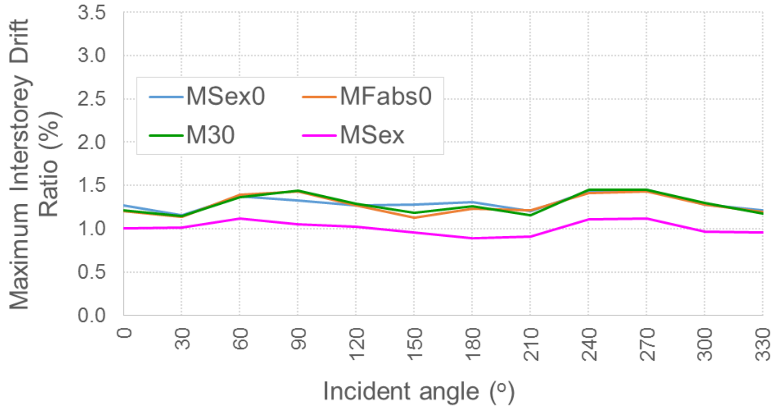


Fig. 4 - Variation of MIDR with incident angle for performance level SD

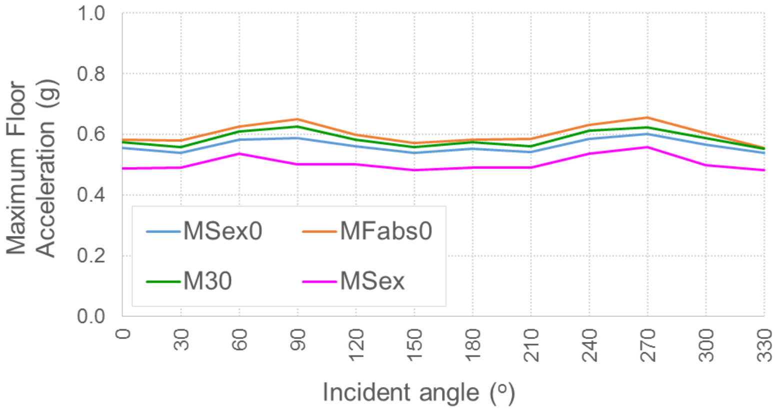


Fig. 5 - Variation of MFA with incident angle for performance level SD



 

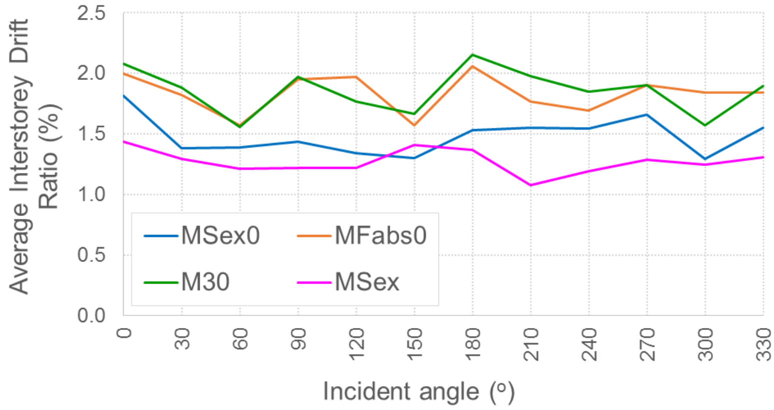


Fig. 6 - Variation of AIDR with incident angle for performance level NC

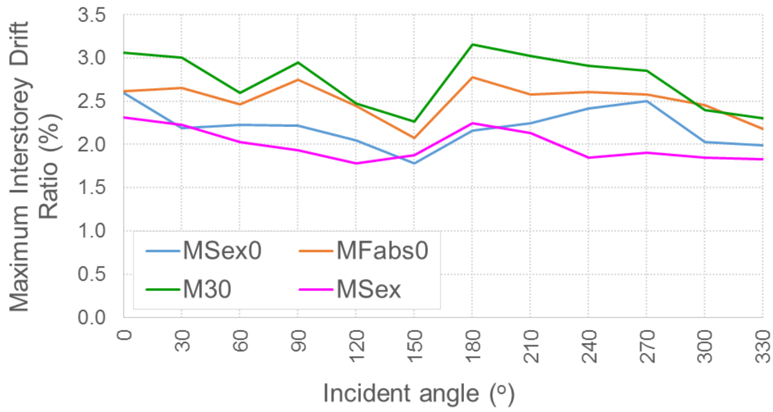


Fig. 7 - Variation of MIDR with incident angle for performance level NC

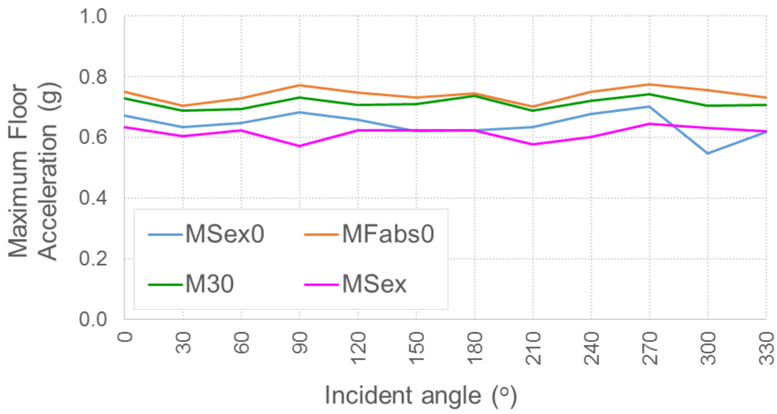


Fig. 8 - Variation of MFA with incident angle for performance level NC

We can see that procedure $MS_{ex}$ produced less secere damage (smaller values of the three damage measures considered in the study) than the other three procedures ($MS_{ex}0$, $MF_{abs}0$ and M30). The difference between the damage indices' values produced by $MS_{ex}$ and $MS_{ex}0$, $MF_{abs}0$ and M30 depends on the incident angle, the damage index and the performance level. Note, for example, that, for performance level SD, for the majority of the incident angles the use of procedure $MS_{ex}$ led to MIDR values close o 1.0%, whereas procedures $MS_{ex}0$, $MF_{abs}0$ and M30 produced values which can reach even 1.5% (Fig. 4). Similarly, for performance level NC, procedure $MS_{ex}$ led to MIDR values between 1.8% and 2.3%, whereas procedures $MS_{ex}0$, $MF_{abs}0$ and M30 produced values which can reach even 3.2% (Fig. 7).

A comparison among proceduresbased on seismic effects produced by accelerograms applied along the structural axes ($MS_{ex}0$, $MF_{abs}0$ and M30) fails to indicate a certain trend concerning the procedure that leads to the best seismic performance of the building, since it strongly depends on the incident angle, the damage index and the performance level. In general, we see that the difference between the results produced by these procedures is larger in case of AIDR and MIDR for performance level NC. However, the incident angle influences the effectiveness of the abovementioned methods, for example note that in case of performance level NC procedure $MF_{abs}0$ led to larger values of AIDR than the corresponding values for procedure M30 for angles 120°, 270° and 300°, whereas for the rest angles M30 produced more severe damage than $MF_{abs}0$ (Fig. 6).

Also, we see that the procedures $MS_{ex}0$, $MF_{abs}0$ and M30 produced results which depend on the seismic incident angle, especialy in the case of AIDR and MIDR for performance level NC. However, the influence of the seismic motion's orientation is smaller in most cases of method $MS_{ex}$. For example, the average value of MIDR produced by $MS_{ex}0$ procedure for performace level NC (Fig.7) ranges between 1.8% and 2.6% for incident angle 240° and 0° respectively. We should notice that if the design is performed by using the $MS_{ex}$ procedure, the impact of seismic incident angle on the damage state of the building is smaller (values of MIDR range between 1.8% and 2.2%).

Fig. 9 illustrates the total weight of longitudinal reinforcement for the four procedures examined in the present paper. More specifically, the figure presents the average reinforcement for the seven earthquake records, computed seperately for the columns, the beams, as well as for both of them (total weight). It can be seen that procedure $MS_{ex}$ led to the largest reinforcement weight in case of the beams, whereas in case of the columns, method $MF_{abs}0$ followed by $MS_{ex}$ produced the largest weight. Regarding the total weight, we can see in Fig. 9 that procedures $MS_{ex}0$ and M30 led to the smallest reinforcement weight while $MF_{abs}0$ and $MS_{ex}$ produced the largest weight of reinforcement. Note that the reinforcement weight produced by $MS_{ex}0$ and M30 is 17% and 11% smaller than the reinforcement produced by $MS_{ex}$ respectively. Concerning procedures $MF_{abs}0$ and $MS_{ex}$, we see that they led to almost the same reinforcement weight. However, from the analyses conducted in the present paper, it is shown that when the building is designed using procedure $MS_{ex}$ the inelastic performance of the structure is much better. The superiority of procedure $MS_{ex}$ regarding the seismic performance of the structure is attributed not only to the larger reinforcement ratios but also to the better distribution of the required reinforcement among the building cross sections.

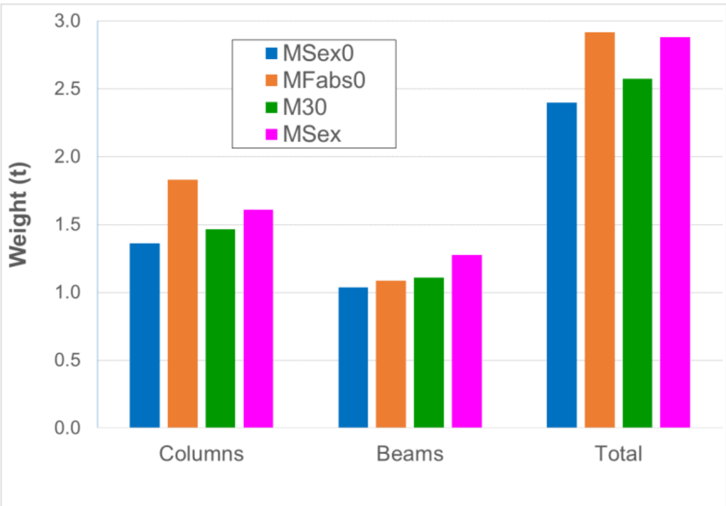


Fig. 9 - Total weight of the longitudinal reinforcement

## 5. Conclusions

If the reinforcement in R/C members depends on more than one response parameter, code provisions do not clear define how to choose the combination of sectional forces that produce the reinforcing steel ratio. In the present paper four different procedures for the appropriate selection of the sectional forces needed for the design of R/C buildings within the context of linear response history analysis are evaluated. The evaluation is performed based on three global damage measures. Moreover, in order to investigate the influence of the seismic intensity on the inelastic structural response of the building two different earthquake intensity levels are considered.The comparative assessment of the results leads to the following conclusions:

- The inelastic response of the building is affected by the procedure used to select the design sectional forces in R/C frame elements.
- For both the intensity levels considered in the present study, the procedure $MS_{ex}$ leads to better seismic performance of the building (smaller values of damage measures) compared to the performance produced by the other three procedures.
- A comparison among procedures based on seismic effects produced by accelerograms applied along the structural axes ($MS_{ex}0$, $MF_{abs}0$ and M30) fails to indicate a certain trend concerning the procedure that leads to the best seismic performance of the building, since it depends on the earthquake intensity level and the incident angle.
- The incident angle affects the seismic performance of the building when the procedures $MS_{ex}0$, $MF_{abs}0$ and M30 are used to select the sets of internal forces. However, the influence of the incident angle on the damage state of the building is smaller for the $MS_{ex}$ procedure.
- Procedures $MS_{ex}0$ and M30 lead to the smallest reinforcement weight while $MF_{abs}0$ and $MS_{ex}$ produce the largest weight of reinforcement. The procedures $MF_{abs}0$ and $MS_{ex}$ produce about the same weight of reinforcement. The superiority of procedure $MS_{ex}$ to the other three procedures regarding the seismic performance of the structure is attributed not only to the larger reinforcement ratios but also to the better distribution of the required reinforcement among the building cross sections.
- The procedure $MS_{ex}$ leads to the better seismic performance independend of the incident angle.

The design procedure presented in the present paper was applied in a R/C building. However, the methodology is general and can be applied to any form of structures (e.g.: bridge piers, dams, etc.)

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