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Implementation of Displacement Coefficient method for seismic assessment of buildings built on soft soil sites

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ABSTRACT

This paper presents the results of an investigation aimed at extending the Coefficient Method for the seismic assessment of existing buildings built on very soft soil conditions. In the first part of this investigation, the lateral displacement response of four steel frames and six reinforced concrete frames under a set of 20 earthquake ground motions recorded on very soft soil sites of the old bed-lake of Mexico City is investigated. It is shown that the seismic response of the buildings strongly depends on the ratio of the first-mode period of vibration of the structure to the predominant period of the ground motion (T/T_g). In the second part of this study, a Displacement Coefficient method approach is employed for obtaining estimates of maximum inelastic roof displacement demands. Error statistics indicates that the Coefficient Method provides reasonably good estimates.

1. Introduction

Modern performance-based seismic assessment procedures for existing structures are based on the evaluation of: (a) the structure-specific lateral deformation capacity, and (b) the earthquake-induced displacement demand. Among them are the nonlinear static procedures discussed in FEMA-273/274 [1], FEMA 356 [2], and FEMA 440 [3] recommendations as well as the ASCE 41-06 [4] standard in the United States. They have become popular among American practicing engineers due to their simplicity and ability to provide useful insight regarding the expected performance of earthquake-resistant structures. In particular, the so-called Coefficient Method is employed for estimating the maximum roof (target) displacement demands for simplified performance-based assessment of existing buildings. Therefore, several studies have focused their attention on evaluating the ability of the Coefficient Method for predicting the maximum roof displacement demand of existing buildings (e.g. [5–8]), but considering only existing buildings subjected to far-field or near-fault earthquake ground motions recorded on firm soil site conditions. However, there is still a need of evaluating the ability of Coefficient-based methods for predicting the target inelastic displacement demand of existing buildings built on soft soil sites, such as the bed-lake zone of Mexico City or the San Francisco Bay Area, since significant structural damage has been reported in buildings placed in this type of soil when subjected to earthquake ground shaking (e.g. [9]).

The primary objective of the research reported in this paper is to evaluate the effectiveness of a Coefficient-based Method for estimating peak roof inelastic displacement demands of steel and reinforced concrete framed-buildings subjected to soft-soil earthquake ground motions. The evaluated method aims to provide initial screening of building performance during the first stage of seismic evaluation, but it does not aim to substitute a detailed seismic evaluation of the building under consideration (e.g. using dynamic nonlinear time–history analyses). In addition, it should be noted that the method is constrained to case-study buildings that are fixed at their base, which implies that soil–structure interaction effects are negligible, and the influence of the soft soil site conditions in the seismic response is taken into account through the frequency content of the earthquake ground motions.

2. Review of Displacement Coefficient method

2.1. Background

The pioneering interest of providing simplified procedures for estimating maximum lateral inelastic displacement demands (e.g. roof and maximum over all stories) for mid-rise reinforced concrete (RC) building structures dates back to mid-70s by Shibata and Sozen [10]. It should be noted that this interest was beyond providing estimates of nonlinear displacement response of simple structures, which can be modeled as single-degree-of-freedom (SDOF) systems, but primarily to provide a tool for practicing design engineers to meet framed-building lateral stiffness required to avoid undesirable level of damage related to threshold

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maximum inter-story drift demands [11]. For instance, the simplified method outlined by Shimazaki and Sozen [12] suggested that maximum inelastic roof drift demand can be obtained from the modification of spectral elastic displacement ordinates through two modification factors that represent: (1) the normalized inelastic displacement obtained from a constant lateral-strength SDOF oscillator with respect to its elastic counterpart; and (2) the mode participation coefficient. To the author's knowledge, this procedure could be named as the first "Displacement modification" method. Years later, Krawinkler and his co-workers [13–15] proposed a seismic design procedure for framed-building and wall structures based on achieving target ductility capacity for collapse safety, where a key step consists in including modification factors to obtain estimates of inelastic demands from elastic strength and displacement spectral ordinates. Similarly to Sozen's approach, modification factors proposed in Krawinkler's approach took into account the relationship between the elastic displacement response of a SDOF to a MDOF and the relationship of the inelastic to the elastic displacement demand of SDOF systems, but the effect of hysteretic behavior in the nonlinear displacement response of SDOF systems was considered as an additional modification factor [16]. It should be mentioned that other "Displacement modification" approaches have been proposed for the preliminary design of building structures, where an estimation of maximum roof and inter-story drift demands are of primary importance, such as those introduced by Qi and Moehle [17] and Miranda [18].

Based on the research developed by Krawinkler and his research group [13–16] and with the aim of providing a simplified tool for seismic assessment and rehabilitation of existing structures based on displacement-based procedures, the FEMA 273 [1] and FEMA-356 [2] guidelines introduced the Nonlinear Static Procedure (NSP) to obtain estimates of the seismic performance of buildings. The core concept was to apply monotonically increasing lateral forces to a mathematical model of the building under consideration until either a target displacement is exceeded or the building collapses. Based on extensive analytical studies, improvements to the estimation of the target displacement were proposed in FEMA 440 document [3] and later incorporated in the ASCE 41-06 [4] Standard Seismic Rehabilitation of Existing Buildings. In the ASCE 41-06 [4] Standard, the target roof displacement can be obtained as follows:

$$\delta_t = C_0 C_1 C_2 S_a \left(\frac{T_e^2}{4\pi^2}\right) g \tag{1}$$

where C_0 is a modification factor that relates spectral displacement and likely building roof displacement, C_1 is a modification factor to relate expected maximum inelastic displacements to displacements calculated from a linear elastic analysis, C_2 is a modification factor to represent the effect of hysteretic behavior on the maximum displacement response, S_a is the response spectrum acceleration at the effective fundamental vibration period and damping ratio of the building under consideration, and T_e is the effective fundamental period of the building. Particularly, the following expression for estimating coefficient C_1, is included in the ASCE 41-06 [4] Standard:

$$C_1 = \begin{cases} 1.0 & T_e > 1.0s \\ 1.0 + \frac{R-1}{aT_e^2} & 0.2s < T_e \le 1.0s \\ 1.0 + \frac{R-1}{0.04a} & T_e \le 0.2s \end{cases} \tag{2}$$

where a is a site-depended coefficient (e.g. equal to 130 for site class A and B, 90 for site class C, and 60 for site classes D, E, and F) and R is the strength ratio defined as the ratio of the elastic strength demand to calculated yield strength coefficient, which also represents the ground motion intensity with respect of the lateral strength of the buildings under consideration (i.e. a relative lateral strength measure). Likewise, the coefficient C_2 can be computed as follows:

$$C_2 = \begin{cases} 1.0 & T_e > 0.7s \\ 1 + \frac{1}{b}\left(\frac{R-1}{T_e}\right)^c & T_e \le 0.7s \end{cases} \tag{3}$$

where b and c take values of 800 and 2. It should be noted that FEMA 440 recommendations [4] recognized the inherent uncertainty in the estimation of the target roof displacement. In particular, FEMA 440 [4] stated that "When interpreting results and assessing structural performance, engineers should consider the implications of such uncertainties. For example, the expression can be used with $a = 60$ for softer sites (class E and F) to estimate displacements, but it is less reliable due to the very high dispersion of results in studies of SDOF oscillators for soft sites.". Therefore, it is of particular interest to evaluate the accuracy of (1) in estimating maximum roof inelastic displacement demands of existing buildings when subjected to ground motions recorded in soft soil sites.

2.2. Coefficient C_1 for soft soil sites

A key factor in the estimation of the target displacement in Eq. (1) is the coefficient C_1, which is also known as the inelastic displacement ratio, C_R, in the literature [e.g. 19,20]. Previous studies developed by the main author had shown that the record-to-record variability in the estimation of inelastic displacement ratios for soft soil sites could be reduced if C_R ratios are computed from normalized period of vibration with respect to the predominant period of the ground motion, T/T_g [20]. However, it should be noted that the spectral shape of C_R computed from this approach significantly differs from that computed for firm soil sites as shown in Fig. 1. As a consequence, the functional form of Eq. (2) is not suitable for providing estimates of coefficient C_1. To remedy this issue, Ruiz-García and Miranda [20] suggested the following functional form to obtain estimates of C_R (i.e. coefficient C_1 in Eq. (1)) to be used in a Displacement Coefficient approach for performance-based assessment of existing buildings.

$$\begin{aligned}\overline{C}_R = {} & \theta_1 + (R-1)\left[\frac{1}{\theta_2 \cdot (T/T_g)^2}\right] \\ & + \theta_3 \cdot (T_g/T) \cdot \exp[-4.5 \cdot \{\ln(T/T_g - 0.05)\}^2] \\ & + \theta_4 \cdot (T_g/T) \cdot \exp[\theta_5 \cdot \{\ln(T/T_g + 0.67)\}^2]\end{aligned} \tag{4}$$

where T is the period of vibration, T_g is the predominant period of the ground motion and θ_1, θ_2, θ_3, θ_4,and θ_5 are parameters whose estimates depend on the type of soft soil site (e.g. old-bed lake zone of Mexico City or the bay-mud area of San Francisco) and they can be obtained through nonlinear regression analysis techniques. Parameter estimates that can be used for buildings built on soft soil sites of Mexico City can be found in Ruiz-García and Miranda [20].

3. Framed buildings and earthquake ground motions considered in this study

3.1. Building frames and modeling assumptions

Two families of regular framed-buildings were considered in this investigation. All buildings were assumed to be designed for office occupancy and located in the lake-bed zone of Mexico City. The first family includes four three-bay steel buildings having 4, 6, 8 and 10 stories. Fig. 2a shows the plan view of the steel buildings. All buildings were designed by an experienced structural engineering office to satisfy the 2004 Edition of the Mexico City Building Construction Code [21]. Moment-resisting frames were provided in both the longitudinal and transverse direction, while

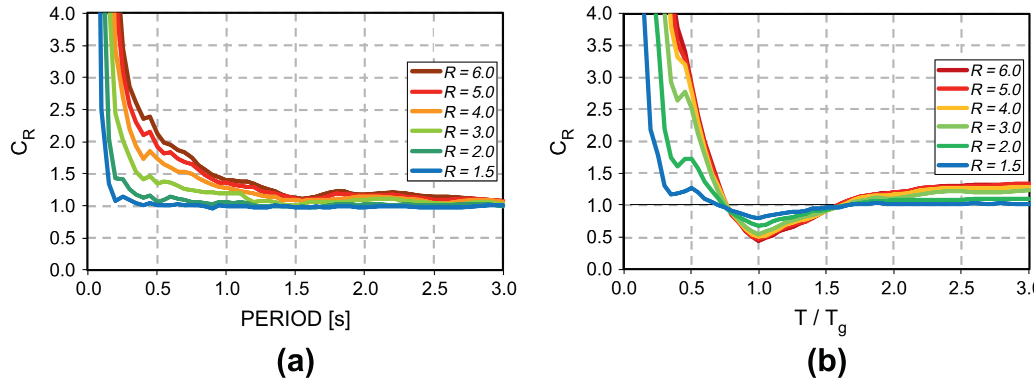

Fig. 1. Mean inelastic displacement ratio (i.e. coefficient C_1) computed for: (a) firm soil sites (adapted from: [19], (b) soft soil sites (adapted from: [20]).

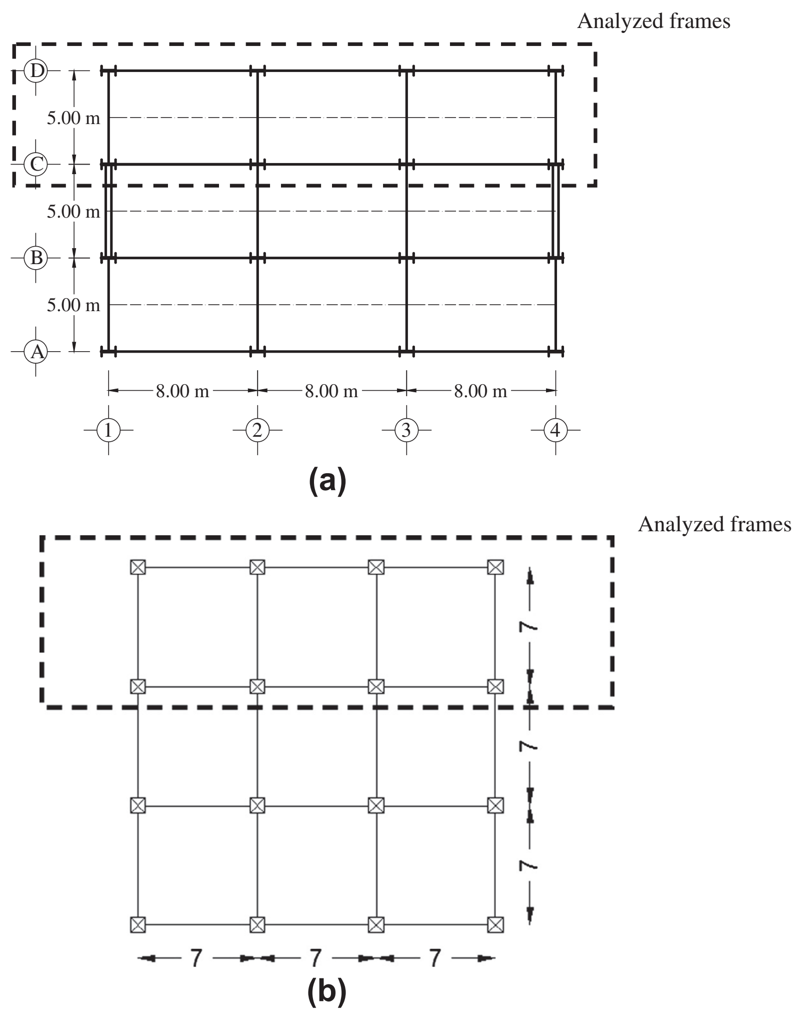

Fig. 2. Plan view of case-study buildings (units in meters): (a) steel buildings, (b) RC buildings.

additional eccentric braces (EB) where incorporated in the transverse direction for drift control since the weak-axis of the columns is oriented in this direction. However, it should be pointed out that the goal of this, and other investigations (e.g. [22]) was to access the seismic response of the moment-resisting steel frames in the longitudinal direction. Elastic acceleration design spectrum ordinates were reduced by a response modification factor equal to 2 in both directions, which takes into account the ability of the structure to undergo inelastic deformations, without consideration of structural overstrength, but with limited ductility. Thus, an equivalent static linear analysis, which is commonly used in the Mexican design practice, assuming a triangle inverted distribution of code-specified base shear was employed for sizing the frame members. Further information on the steel frames can be found in [22].

The second family comprises six three-bay reinforced concrete (RC) buildings with 4, 6, 8, 10, 12, and 16 number of stories. Fig. 2b shows the plan view of the RC buildings. All buildings were designed to satisfy the 1997 Edition of the Mexico City Building

Construction Code [23]. Moment-resisting frames were considered in both directions following Mexican design practice. Design acceleration spectrum was obtained from reducing the elastic acceleration spectrum by a response modification factor equal to 4 in both directions. Detailed information of the design process and assumptions can be found in Teran-Gilmore [24]. To provide a context with respect to American Standards, the steel frames and RC frames could be considered as ordinary and special moment-resisting frames, respectively.

3.2. Modeling assumptions for nonlinear analyses

All steel and RC buildings were analyzed with the nonlinear dynamic analysis computer program RUAUMOKO [25]. Only half of each building was modeled due to symmetry in the building's plan. An exterior and interior frame were modeled as two-dimensional centerline models, assuming fixed columns which imply that base flexibility and soil–structure interaction was neglected. Both frames were attached through rigid frame elements to experience the same lateral deformation at each floor assuming a rigid diaphragm. It should be mentioned that in Mexican design practice, all frames in the same direction are designed as moment-resisting frames which carry the gravity loads proportional to their tributary area (i.e. interior columns and beams are not considered only as load-carrying elements).

Beams and columns in the steel frames were modeled as frame elements which concentrate their inelastic response in plastic hinges located at their ends (i.e. lumped plasticity nonlinear frame elements). Plastic hinge length was assumed as 80% and 50% of the section depth for the steel and RC members, respectively. Non-degrading quasi-elastoplastic (i.e. with strain-hardening ratio close to 0% to avoid numerical instabilities) moment–curvature relationship that considers axial load–flexural bending interaction was considered to model the hysteretic behavior of the steel columns, while the steel beam's behavior was modeled through a quasi-elastoplastic moment–curvature relationship (Fig. 3a). However, slab contribution to the beam's bending capacity was neglected in this study. On the other hand, hysteretic behavior of RC columns was modeled with Takeda-type moment–curvature relationship, with very small strain-hardening ratio to avoid numerical instabilities, and axial load–flexural bending interaction whereas RC beams was modeled with Takeda-type moment–curvature relationship (Fig. 3b).

For dynamic analyses, second order effects were explicitly considered (i.e. large displacement analysis). Mass- and-stiffness proportional Rayleigh damping was considered for the analysis, where a 5% of critical damping was assigned to the first two modes of vibration of the frames. Before conducting nonlinear time–history analyses, modal and nonlinear static (pushover) analyses of each frame were performed to obtain relevant dynamic and mechanical characteristics such as the fundamental period of vibration, T_1, the building's yield coefficient (i.e. base shear normalized with respect to the building's weight), C_y, roof drift at yielding, θ_y, and the normalized modal participation factor at the roof, $\Gamma_{1\phi 1,roof}$, as it is reported in Tables 1 and 2. It should be mentioned that pushover analysis was conducted with the software RUAUMOKO [25] by using a slow ramp loading function assuming a triangular-inverted loading distribution as prescribed in the Mexican seismic design standards.

Finally, it should be mentioned the numerical modeling of the case-study frames for computing their nonlinear response followed well-known modeling strategies (e.g. lumped-plasticity elements, Rayleigh damping, rigid diaphragm assumption, etc.), but calibration of the analytical models was beyond the scope of the paper.

3.3. Set of earthquake ground motion records from soft soil sites

The frame models considered in the case studies were subjected to a set of 20 narrow-band ground motions recorded at soft soil sites of Mexico City from 5 historical earthquakes, which are included in the Mexican Database of Strong Motions [26]. The seismic events had seismic magnitudes ranging between 6.3 and 8.1, with epicenters located at distances about 300 km, or more, from Mexico City. Important features of the earthquake ground motions are summarized in Table 3.

Particularly, it should be noted that the predominant period of the ground motion, T_g, was computed as the period at which the maximum ordinate of a 5% elastic damped relative velocity spectrum occurs [27]. For the soft soil deposits of Mexico City, T_g has been found to be closely related with the predominant period of the soil deposit computed from one-dimensional elastic models assuming that the response of the soil deposit is dominated by vertically propagating shear waves in a layered deposit and the second mode of vibration is approximately one-third of the fundamental period of vibration [28].

4. Response under soft soil records

4.1. Evaluation of the lateral strength ratio

At a first stage, the relative lateral strength was computed for each building under each earthquake ground motion considered in Table 3, which is defined as follows:

$$R = \frac{S_a(T_1)}{V_{b,y}/W} \quad (5)$$

where $S_a(T_1)$ is the spectral acceleration corresponding to building's fundamental period, $V_{b,y}$ is the base shear corresponding at yielding and W is the total weight of the building. This initial evaluation showed that each of the steel frames lead to relative lateral strength ratios smaller than one under each earthquake ground motion, as

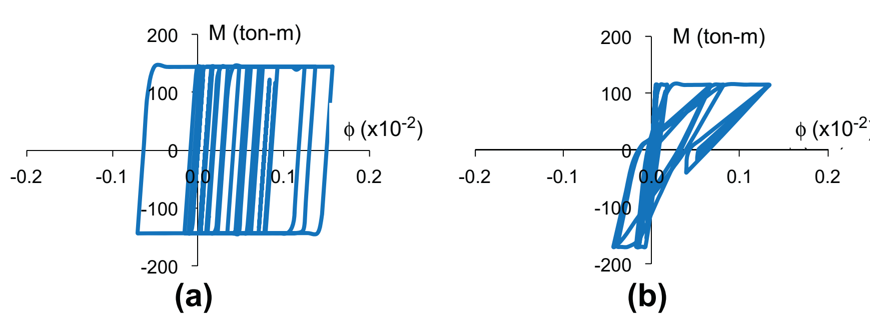

Fig. 3. Element hysteretic behavior considered in this study: (a) quasi-elastoplastic model (steel frames), (b) Takeda model (RC frames).

Table 1
Properties of steel frames considered in this investigation.

Model	T_1 (s)	C_y	θ_y (%)	$\Gamma_1\phi_{1,roof}$
A-4N	0.74	0.72	1.07	1.32
A-6N	0.88	0.65	0.95	1.36
A-8N	0.93	0.64	0.79	1.38
A-10N	1.06	0.59	0.74	1.37

Table 2
Properties of RC frames considered in this investigation.

Model	T_1 (s)	C_y	θ_y (%)	$\Gamma_1\phi_{1,roof}$
C-4N	0.81	0.32	0.54	1.25
C-6N	1.14	0.22	0.51	1.31
C-8N	1.40	0.19	0.62	1.32
C-10N	1.40	0.19	0.54	1.39
C-12N	1.41	0.21	0.49	1.36
C-16N	1.74	0.20	0.58	1.43

shown in Fig. 4a, which implies that the frames did not experience nonlinear behavior. Similarly, most of the relative lateral strength ratios computed for the RC frames were smaller than one, as shown in Fig. 4b, since they have smaller lateral yield strength coefficient than the steel frames. It should be noted that R values larger than one were mainly triggered by the records gathered in the SCT accelerographic station (i.e. the frames experienced nonlinear behavior). Since the objective of this study is to evaluate the nonlinear maximum roof drift demand in the case-study buildings, it was decided to scale up the acceleration spectral ordinates in such a manner that all buildings reach a lateral strength ratio equal to two (i.e. keeping $V_{b,y}/W$ constant in Eq. (5)).

4.2. Influence of the predominant period of the ground motion

Previous studies have shown that inelastic displacement ratio spectra computed for soft soil sites follow a different trend than that computed for firm soil sites [20]. For instance, three spectral regions with distinctively different characteristics can be identified in the mean C_R spectra for soft soil sites (i.e. for T/T_g ratios smaller than about 0.75, for T/T_g ratios between about 0.75 and 1.55, and for T/T_g ratios larger than 1.55) shown in Fig. 1b, while mean C_R spectra for firm sites exhibit two spectral regions. More specifically for earthquake ground motions recorded on soft soil sites of Mexico City (Fig. 1b), maximum inelastic displacements are, on average, larger than maximum elastic displacements in the first region corresponding to T/T_g ratios smaller than approximately 0.75. In this spectral region, inelastic displacement ratios increase as the lateral strength ratio increases and as normalized periods T/T_g decrease. The second spectral region corresponds to systems with periods of vibration relatively close to the predominant period of the ground motion where peak lateral deformation of inelastic systems are, on average, smaller than peak lateral deformations of elastic systems, which might be

Table 3
Earthquake ground motions considered in this investigation.

Nomenclature	Station ID	Station name	Date	Magnitude (Ms)	Component	PGA (cm/s^2)	T_g (s)
SCT10DIEW	SC	SCT	10/12/1994	6.3	EW	15.0	1.89
SCT19SEEW	SC	SCT	19/09/1985	8.1	EW	167.9	2.06
SCT19SENS	SC	SCT	19/07/1985	8.1	NS	97.9	2.07
4424OCEW	44	U. Colonia IMSS	24/10/1993	6.6	EW	15.0	1.34
4424OCNS	44	U. Colonia IMSS	24/10/1993	6.6	NS	12.2	1.52
5524OCNS	55	Tlatelolco	24/10/1993	6.6	NS	8.3	1.35
RO14SEEW	RO	Roma	14/09/1995	7.1	EW	37.4	1.31
RO14SENS	RO	Roma	14/09/1995	7.1	NS	28.6	1.40
RO25ABEW	RO	Roma	25/04/1989	6.9	EW	54.7	1.27
RO25ABNS	RO	Roma	25/04/1989	6.9	NS	45.4	1.53
RO10DIEW	RO	Roma	10/12/1994	6.3	EW	12.0	1.39
RO10DINS	RO	Roma	10/12/1994	6.3	NS	14.2	1.32
4425ABEW	44	U. Colonia IMSS	25/04/1989	6.9	EW	39.6	1.28
5325ABEW	53	San Simón	25/04/1989	6.9	EW	30.5	1.56
4425ABNS	44	U. Colonia IMSS	25/04/1989	6.9	NS	52.3	1.36
5325ABNS	53	San Simón	25/04/1989	6.9	NS	39.7	1.40
2925ABEW	29	Villa del Mar	25/04/1989	6.9	EW	46.5	2.96
2925ABNS	29	Villa del Mar	25/04/1989	6.9	NS	49.4	2.96
4325ABNS	43	Jamaica	25/04/1989	6.9	NS	35.2	3.04
4825ABEW	48	Rodolfo Menéndez	25/04/1989	6.9	EW	47.7	2.89

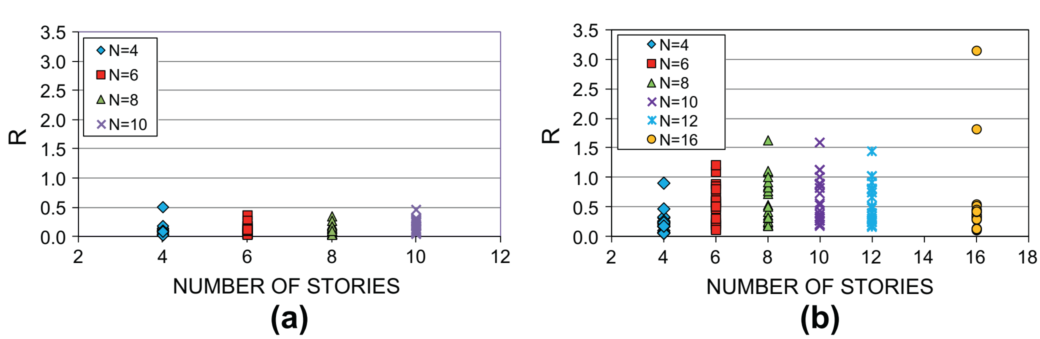

Fig. 4. Initial relative lateral strength ratio (i.e. from as-recorded earthquake ground motions) for the set of frames considered in this study: (a) steel frames, (b) RC frames.

counterintuitive since this trend is not observed in the mean C_R spectra for firm sites. A third region, corresponding to periods of vibration approximately 1.55 times larger than the predominant period of the ground motion, is characterized by peak displacements in inelastic systems being on average slightly greater than peak elastic displacement demands. Similar trend has been observed for C_R spectra computed from earthquake ground motions recorded in the San Francisco Bay area [20], but with different T/T_g boundaries that define the spectral regions. Therefore, it is interesting to review if this trend is also observed in the lateral displacement response of multi-degree-of-freedom systems. For this task, each of the case-study buildings was subjected to each individual record reported in Table 3 and, thus, each response corresponds to a particular T/T_g ratio. It was noted that the trend observed from SDOF studies is still preserved in the response of the frame models, particularly that there is a spectral region where peak inelastic displacement demands become smaller than their elastic counterparts. Typical plots are shown in Figs. 5 and 6 for steel and RC frames, respectively.

In order to find an explanation, both linear and nonlinear time–history responses as well as selected member moment–curvature hysteretic response were examined for selected T/T_g ratios. It was consistently found that when T_g is much longer than T, the earthquake ground motion leads to large permanent curvature deformations after first yielding in the members due to the long duration of the ground motion waveformarts and, thus, peak inelastic displacements become larger than peak elastic displacements. Conversely, when T became closer to T_g, the ground motion waveformarts in the intense phase induce large number of cycles, which leads to smaller permanent curvature deformations after first yielding in the members due to a self-centering hysteretic behavior in the members. To illustrate this explanation, Figs. 7 and 8 show typical member moment–curvature hysteretic responses recorded in a beam of the first story of A-10N and C-8N frames, respectively, when subjected to two records with very different T_g In addition, Figs. 9 and 10 shows the corresponding the roof lateral displacement time–history response computed for both frames.

4.3. Evaluation of C_R for MDOF systems

Once reviewed that the maximum elastic displacement demand could be larger than maximum inelastic displacement demands, the following task in this investigation was to assess if the ratios of maximum inelastic roof drift demand to maximum roof elastic drift demand computed from the case-study buildings also follow similar trends than those observed from SDOF studies. Therefore, Fig. 11 shows the $C_{R,M}$ spectra computed for the steel and RC frames. For reference purposes, the C_R trend for lateral strength ratio equal to 2 computed from Eq. (4) is also included in the plot. In spite of the limited range of T/T_g ratios due to the number of frames and earthquake ground motions considered in this study, it can clearly be seen that $C_{R,M}$ decreases as the T/T_g ratios increases and that there is a spectral region where $C_{R,M}$ becomes smaller than one (i.e. peak roof inelastic displacement demands becomes smaller than their elastic counterparts), similarly to the trend observed in SDOF studies. Furthermore, it can also be observed that the dispersion in $C_{R,M}$ increases as the T/T_g ratio decreases, which is consistent with previous results from SDOF systems (e.g. see Fig. 4b in Ruiz-García and Miranda [20]). Therefore, the results generated in this study confirm the general trends highlighted in Ruiz-García and Miranda [20], which suggest that the functional form of Eq. (4) should be employed for the seismic assessment of buildings built on soft soil sites instead of Eq. (2).

Differences in spectral ordinates between $C_{R,M}$ and $\overline{C}_R$ computed from Eq. (4) could be anticipated, since the functional form of $\overline{C}_R$ was fitted from statistical results derived from elastoplastic SDOF systems, which do not include some effects found in the nonlinear response of MDOF systems (e.g. influence of higher modes, frame mechanism, etc.). In addition, it should be recalled that stiffness-degrading hysteretic behavior (i.e. Takeda-type) was considered for modeling the nonlinear moment–curvature member relationship in the RC frames, while member's non-degrading hysteretic behavior was considered in the steel frames. To quantify the MDOF effects not included in $\overline{C}_R$, the ratio of $C_{R,M}$ to $\overline{C}_R$, denoted by C_M, was computed for each frame building and each T/T_g ratio and it is shown in Fig. 12. for each set of frames. It can be seen that C_M

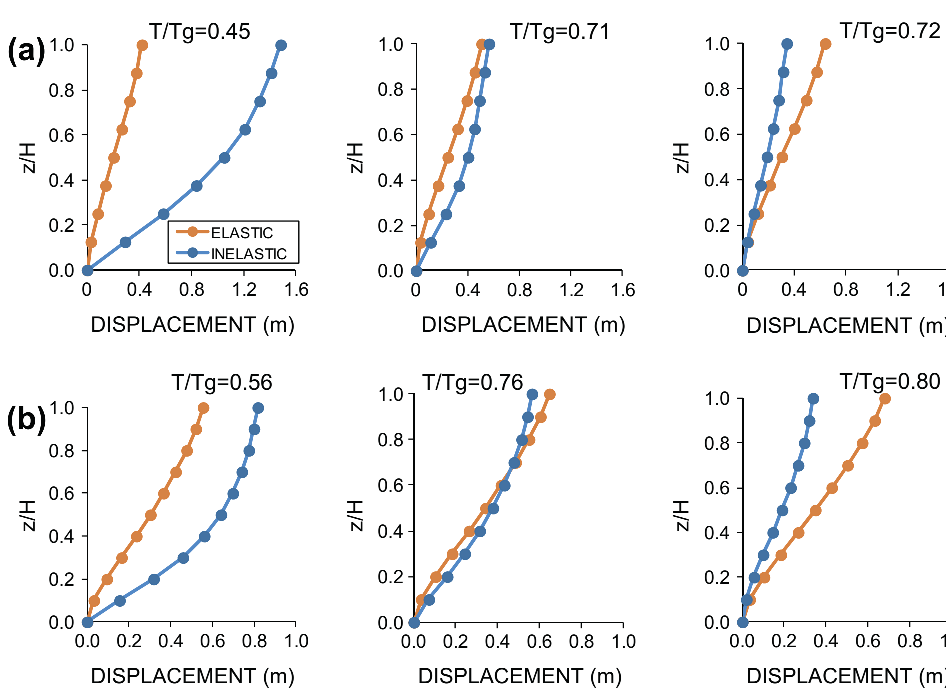

Fig. 5. Lateral displacement profiles computed for steel frames subjected to three records with different predominant period: (a) frame A-8N, (b) frame A-10N.

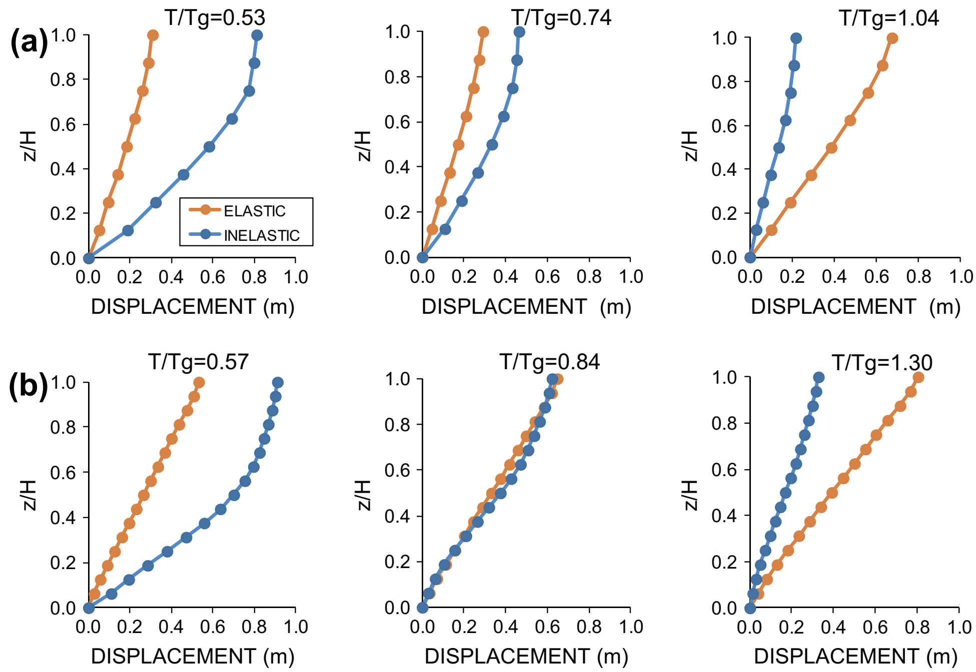

Fig. 6. Lateral displacement profiles computed for two RC frames subjected to three records with different predominant period: (a) frame C-8N, (b) frame C-16N.

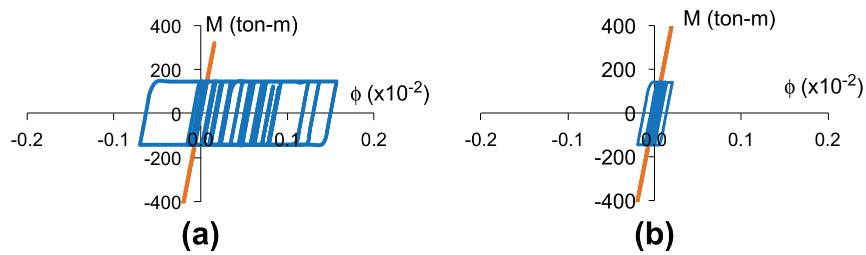

Fig. 7. Moment–curvature response of frame A-10N when subjected to two records with different predominant period: (a) $T/T_g = 0.40$, (b) $T/T_g = 0.83$.

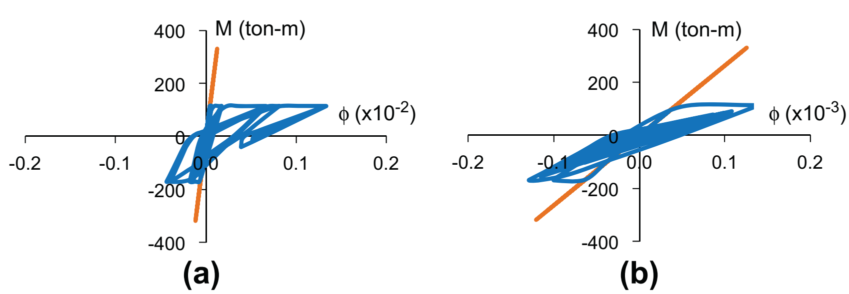

Fig. 8. Moment–curvature response of frame C-8N when subjected to two records with different predominant period: (a) $T/T_g = 0.53$, (b) $T/T_g = 1.04$.

significantly decreases when T/T_g increases for both type of frames, but larger dispersion is seen for the RC frames, which could be attributed to the influence of hysteretic behavior. Recall that the effect of the type of hysteretic behavior has been included in the Coefficient Method as coefficient C_2, which is discussed in the following section.

4.4. Evaluation of coefficient C_2 for soft soil sites

For the family of RC frames, the ratio of peak inelastic roof displacement demand taking into account member's Takeda-type hysteretic behavior to peak inelastic roof displacement demand taking into account member's elastoplastic hysteretic behavior, designated as C_D hereafter, was also computed for each RC frame under each earthquake ground motion. Fig. 13 shows the spectral distribution of C_D, which is characterized for a descending trend as the period ratios decreases. It can be seen that this ratio also follows a similar trend to that reported in Ruiz-García and Miranda [29], which was computed from SDOF systems with different types of degrading hysteretic behaviors, but with spectral ordinates smaller than those shown in Fig. 13. Such difference in spectral ordinates arises from the fact that the nonlinear response of SDOF systems does not capture the influence of the frame mechanism and of other related MDOF effects.

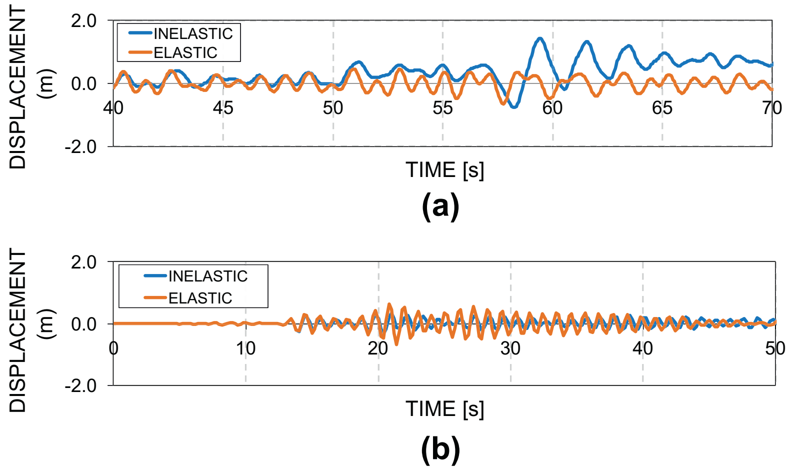

Fig. 9. Roof displacement time–history response of frame A-10N when subjected to two records with different predominant period: (a) $T/T_g = 0.40$, (b) $T/T_g = 0.83$.

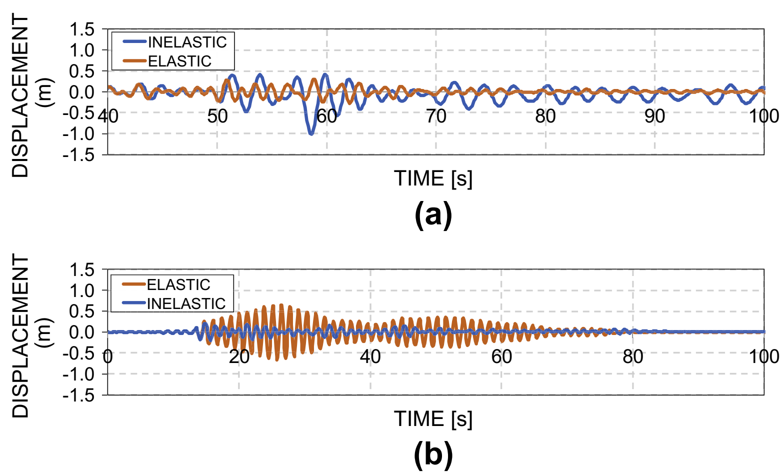

Fig. 10. Roof displacement time–history response of frame C-8N when subjected to two records with different predominant period: (a) $T/T_g = 0.53$, (b) $T/T_g = 1.04$.

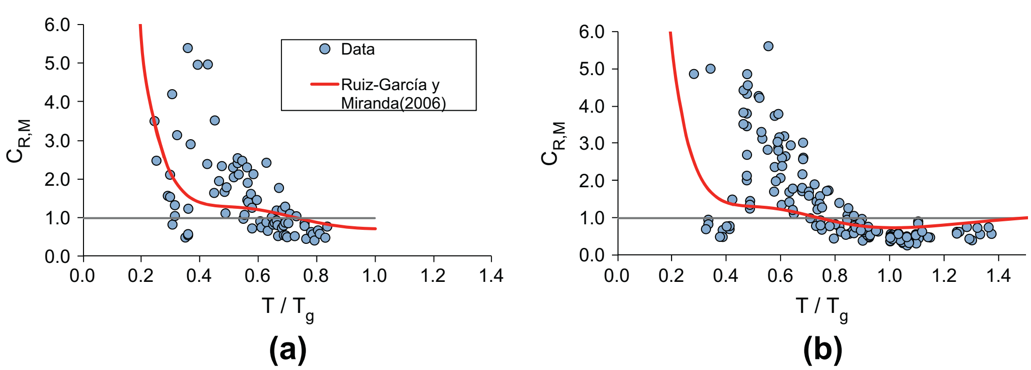

Fig. 11. Comparison of inelastic displacement ratios computed from the MDOF systems under consideration and predicted inelastic displacement ratio using Eq. (4): (a) steel frames, (b) RC frames.

As discussed in Section 2, ASCE 41-06 [4] Standard includes coefficient C_2 (Eq. (3)) to take into account the effect of hysteretic behavior in the estimation of the target (peak) roof displacement demand. Recently, based on analytical studies of degrading MDOF systems, Erduran and Kunnath [7] proposed the following equation to replace Eq. (2):

$$C_2 = (a^{R^b})T^c \quad (6)$$

where estimates of coefficients a, b, and c were obtained from least square curve fitting and they depend on the type of member's hysteretic behavior (e.g. stiffness degrading or low-, moderate-, severe-strength-and-stiffness degrading) and the type of ground

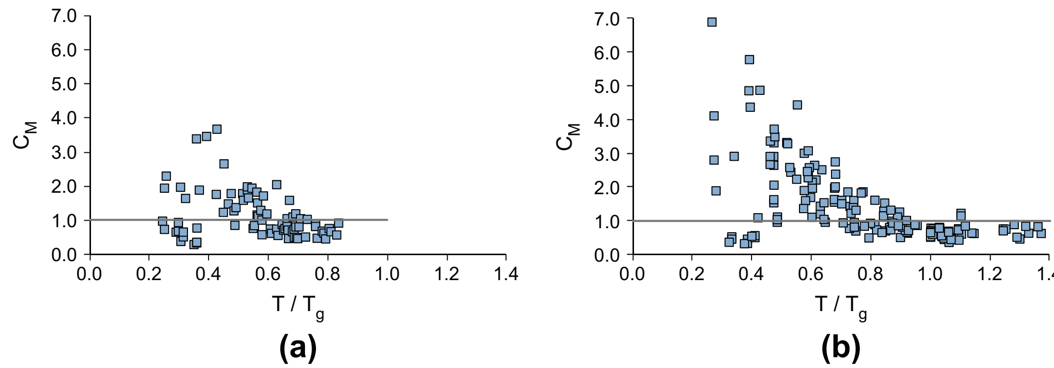

Fig. 12. Ratio of the inelastic displacement ratio computed from MDOF systems: (a) steel frames, (b) RC frames.

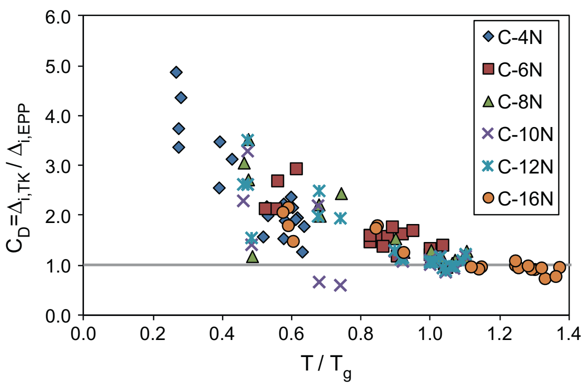

Fig. 13. Spectral distribution of ratios of maximum inelastic roof displacement demand taking into account member's Takeda-type hysteretic behavior to maximum inelastic roof displacement demand (denoted as C_D) taking into account member's elastoplastic hysteretic behavior.

motion (far-field or near-fault). In this investigation, it was successfully verified that the functional forms of Eqs. (3) and (6) follow a similar trend to that shown by empirical data as illustrated in Fig. 14, but the parameter estimates were updated through nonlinear regression analyses since the original values led to smaller ordinates. Therefore, the updated parameters providing the smallest residual standard error for using Eq. (3) are $a = 9.49$ and $b = 1.74$, while for employing Eq. (6) are $a = 1.01$, $b = 2.29$, and $c = -1.20$. Fig. 14 shows the fitted curves of coefficient C_D, or C_2, for soft soil sites. It should be noted that the preceding parameter estimates were obtained for RC frames that include member hysteretic behavior such as that shown in Fig. 3b, but they would change when considering other types of hysteretic behavior.

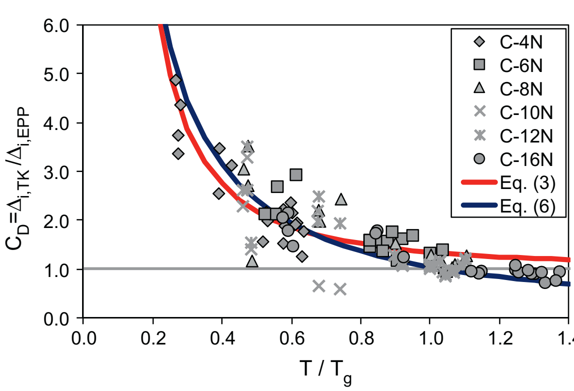

Fig. 14. Comparison of empirical C_D ratios with those computed from fitted Eqs. (3) and (6).

4.5. Evaluation of the coefficient of distortion (COD)

For displacement-based seismic assessment, it is useful to obtain estimates of the maximum interestory drift ratio from the estimated maximum roof drift ratio (e.g. from Eq. (1)). This can be accomplished through the coefficient of distortion (COD), which is defined as the ratio of maximum interstory drift ratio at any story to the maximum roof drift ratio, proposed in [17]. As noted in [e.g. 29,30] the COD depends on the number of stories, the expected failure mechanism and the intensity of the ground motion. Based on the results obtained in this study, the COD corresponding to each set of frame models was computed for each earthquake ground motion and it is plotted as a function of the maximum roof drift ratio $\theta_{i,roof}$ in Fig. 15. In addition, Table 4 reports the sample mean COD for each frame model and its corresponding coefficient of variation (COV).

5. Estimation of maximum roof inelastic displacement through Displacement Coefficient method

After examining the coefficients involved in the Displacement Coefficient method for soft soil sites, this section presents the results of the estimation of maximum roof inelastic displacement demand, $\bar{\delta}_{i,roof}$ for both families of frames. For this purpose, while $\bar{\delta}_{i,roof}$ was computed for each of the steel frames considered in this study through the evaluation of Eq. (7), $\bar{\delta}_{i,roof}$ was computed from Eq. (8) for the RC frames.

$$\bar{\delta}_{i,roof} = C_M \times \Gamma_1 \phi_{1,roof} \times \bar{C}_R \times S_d(T_1) \tag{7}$$

$$\bar{\delta}_{i,roof} = C_D \times \Gamma_1 \phi_{1,roof} \times \bar{C}_R \times S_d(T_1) \tag{8}$$

It should be noted that if soil–structure interaction effects should be considered in the building response, the coefficient $\bar{C}_R$ should have been developed from SDOF systems including base flexibility as well as the case-study buildings should have been modeled with their foundation flexibility. However, these tasks were beyond the scope of this investigation.

Fig. 16 shows a comparison of the maximum roof displacement estimated with the DCM approach with that computed from nonlinear time–history (NLTH) analyses. It can clearly be seen that the simplified approach tends to underestimate, in general, maximum roof displacement demands, which is particularly true for the steel buildings.

In addition, the relative error (i.e. *error* = $(\Delta_{NLTH} - \Delta_{DCM}/\Delta_{NLTH})$, where Δ_{NLTH} is the peak roof displacement computed from

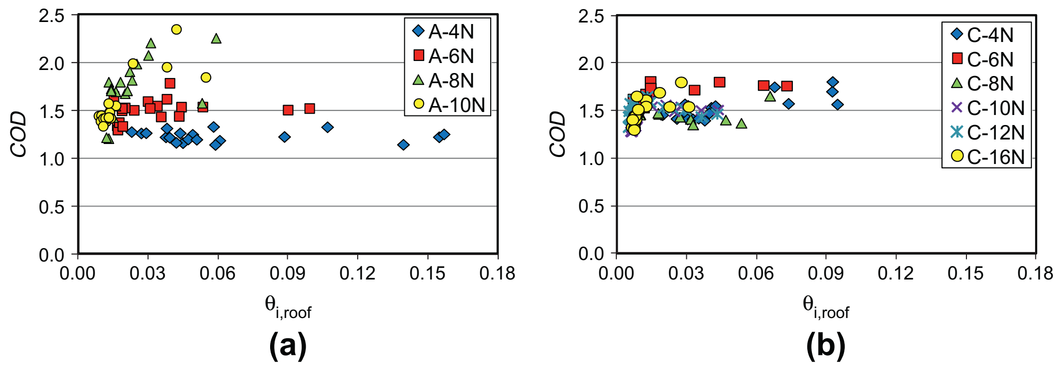

Fig. 15. Coefficient of distortion as a function of maximum roof drift: (a) steel frames, (b) RC frames.

Table 4
Coefficient of distortion computed in this study.

Model	COD	COV
A-4N	1.23	0.05
A-6N	1.51	0.08
A-8N	1.72	0.17
A-10N	1.56	0.17
C-4N	1.54	0.08
C-6N	1.64	0.06
C-8N	1.45	0.04
C-10N	1.41	0.08
C-12N	1.49	0.06
C-16N	1.48	0.09

nonlinear time–history analyses and Δ_{DCM} is the peak roof displacement computed from either Eq. (7) or (8)), was computed for each building when subjected to each of the 20 earthquake ground motions, to quantify the effectiveness of the simplified approach. The spectral distribution of the relative errors is shown in Fig. 17a for the family of steel frames, while a similar plot for the RC frames is illustrated in Fig. 17b. To provide a judgement about the effectiveness of the simplified approach, it is important to distinguish between the positive relative error (i.e. the DCM overestimates maximum roof displacement) and the negative relative error (i.e. the DCM underestimates maximum roof displacement). Hence, it can be observed that the positive relative error computed for the steel frames follows a uniform trend along T/T_g ratios. Unlike steel frames, relative error, either positive or negative, for RC frames

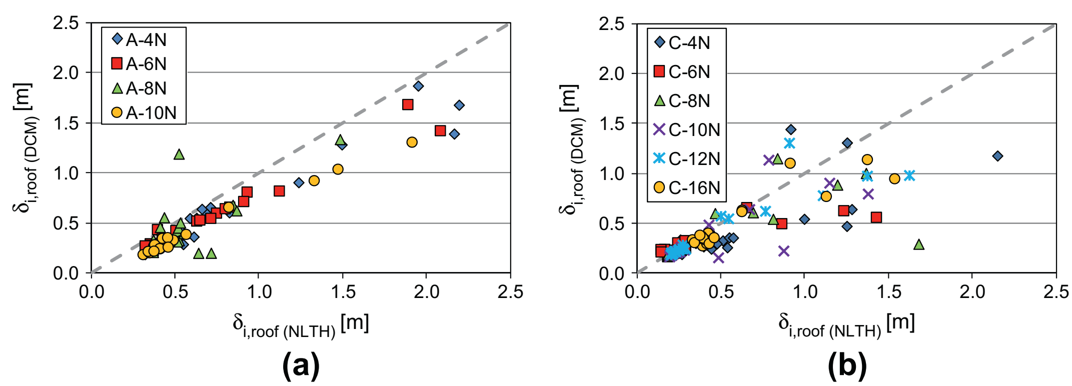

Fig. 16. Comparison of peak roof displacements computed from nonlinear time–history (NLTH) analyses with those computed with Displacement-Coefficient method: (a) steel frames, (b) RC frames.

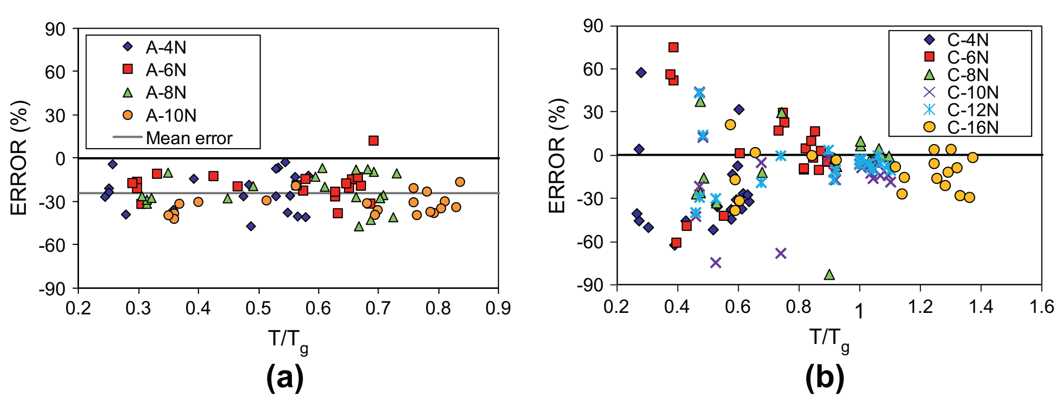

Fig. 17. Relative error of the Displacement Coefficient method to estimate maximum roof displacement demands; (a) steel frames, (b) RC frames.

Table 5
Mean positive and negative relative errors computed in this study.

Frame ID	Mean positive error (%)	Mean negative error (%)
A-4N	0.00	23.29
A-6N	12.39	20.22
A-8N	0.00	22.68
A-10N	0.00	31.82
C-4N	31.52	36.31
C-6N	26.60	20.86
C-8N	15.07	16.01
C-10N	28.15	19.34
C-12N	20.21	12.12
C-16N	6.57	17.11

Table 6
Mean positive and negative relative errors for RC frames computed with fitted Eqs. (3) and (4).

Frame ID	Mean positive error (%)	Mean negative error (%)
C-4N	23.35	37.24
C-6N	14.12	25.32
C-8N	21.27	33.62
C-10N	19.82	26.27
C-12N	23.42	27.78
C-16N	15.92	18.38

seems to decrease as the T/T_g ratios decreases. Mean relative, positive or negative, errors over all T/T_g ratios computed for each case-study frame are reported in Table 5. From this data, the mean positive and negative relative error is 21.35% and 20.29%, respectively, for the RC frames, while the mean negative error for the steel frames is 24.50%. These estimates are reasonably good for preliminary seismic assessment of regular framed-buildings built on soft soil sites, where soil–structure interaction effects are negligible. In addition, relative errors for RC frames taken into account fitted Eq. (3) as C_D are reported in Table 6, where the mean positive and negative relative error is 19.65 and 28.10, respectively. Therefore, using Eqs. (3), properly fitted for soft soil conditions, and (4) still lead to reasonable estimates of maximum roof displacement demands in the RC frames.

6. Conclusions

The Displacement Coefficient method is one of the nonlinear static analyses procedures introduced in the ASCE 41-06 Standard for the seismic evaluation of existing buildings in the US. However, there is a lack of evidence about their ability to estimate target (peak) roof inelastic displacement demands of buildings located in soft soil sites such as the old-bed lake zone of Mexico City. Therefore, the implementation of the Displacement Coefficient method approach for estimation of maximum inelastic roof displacement demand of framed-buildings built on soft soil sites was the main objective of this investigation. For this purpose, the seismic response of four existing steel frames and six existing reinforced concrete frames, designed with the Mexican seismic design standards, was examined as part of this study.

The main results showed that maximum inelastic displacement demands of framed-buildings subjected to earthquake ground accelerations recorded on very soft soil sites strongly depend on the ratio of the first-mode period of vibration of the structure to the predominant period of the ground motion (T/T_g). In particular, it was noted that there is a spectral T/T_g region where maximum roof inelastic displacement demands become smaller than elastic roof displacement demands. Unfortunately, this trend is not captured in the current implementation of the Displacement Coefficient method suggested in the ASCE 41-06 Standard (e.g. through the functional form of coefficient C_1). In addition, it was shown that the regression coefficients in the functional form of coefficient C_2, that takes into account the effect of hysteretic behavior, should be updated when evaluating buildings in soft soil sites since the current equation provides lower estimates.

Estimation of maximum roof inelastic displacement demands through a Displacement Coefficient approach showed reasonably well estimates of the maximum roof inelastic displacement demands obtained from rigorous nonlinear time–history analyses (i.e. with relative errors between 20% and 32% for the steel frames and between 7% and 37% for the RC frames). Therefore, its use seems attractive for the preliminary seismic evaluation of existing buildings located in soft soil sites providing that coefficients C_1 and C_2 are adjusted to take into account this soil condition.

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