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INELASTIC BEHAVIOR OF RC BUILDING CONSIDERING DYNAMIC SOIL-STRUCTURE INTERACTION

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Abstract

Analysis of buildings under seismic actions with flexible base must consider two principal components in the structure displacement, one introduced by structural deformation and the other due to a rigid body behavior. This effect produces that the relation between ductility demand and inelastic capacity of the structure is modified. In addition, consideration of flexible base may change the distribution of internal forces along the structure that could generate variations on the ductility demands over different structural elements. This work explores the behavior of a 10-story regular building with RC moment resisting frames considering the dynamic soil structure interaction. The response of the building with flexible base is compared and contrasted with the rigid base case. The inelastic behavior of the buildings is characterized in terms of ductility capacity and demands. Pushover analysis is used to establish the inelastic capacity parameters. Capacity curves of the building with rigid (fixed) and flexible base are studied. In addition, the comparison of ductility demands is presented. Ductility demands are computed with non linear time history dynamic analysis. Accelerograms used as excitation are scaled to meet design spectral acceleration. Soil-foundation dynamic stiffness (impedance functions) is introduced in the analysis by using a set of springs in horizontal and rocking direction. A mat foundation is considered. A very soft soil with shear wave velocity of Vs=100 m/s is used. Springs stiffness are computed considering the dynamic behavior and properties of the soil-foundation system with a commercial software. Results show that ductility capacity of the soil-structure system is reduced if rigid body displacement components are not eliminated. On the other hand, ductility demands and hysteretic behavior of the global system and local elements are modified due to base flexibility. Some elements experience reduced hysteretic loops when flexible base is considered. Base flexibility produces changes in the relation between yield strength reduction factor and structure ductility demand.

Keywords: Dynamic soil structure interaction; RC buildings; Inelastic behavior; Ductility demands.

1. Introduction

Nowadays the structural design of buildings under seismic actions is based on the concepts of ductility, capacity and performance. This criterion considers that structures will undergo into inelastic behavior when subjected to strong earthquakes, providing an additional energy dissipation source. Design procedures, defined in modern codes around the world are based on the force reductions associated with nonlinear structural behavior. In these procedures, specific collapse mechanisms are assumed (e.g. weak beam-strong column).

Most of the ideas and hypothesis for these design philosophies were developed for systems whose supports are fixed. Under these conditions, the entire displacement of the system is associated with structure deformation, so the ductility is defined as the ratio between maximum and yielding displacements.

Moreover, the dynamic response of structures is controlled by many conditions. Besides structure dynamic properties, seismic excitation characteristics and soil properties play an important roll on structure performance. The interaction between soil and foundation can modify the structure dynamic properties, excitation characteristics and soil behavior. These modifications which arise form soil-foundation joint performance is defined as Dynamic Soil Structure Interaction (DSSI).

Common practice considers DSSI just by the modification of structural period (lengthening) and damping produced by system flexibilization [1]. So, structure will be subjected to a modified spectral acceleration demand. Base shear variation associated with spectral acceleration shift is used to compute changes of remaining response quantities (e.g. displacements, element forces, etc). Even though these DSSI implications are the most acknowledge, other effects of base flexibility could be important also.

Flexible base consideration introduces relative displacements between the structure supports and ground. Soil-structure system displacement includes two principal components, one introduced by structural deformation (u) and other due to a rigid body behavior (u_x and θ) as shown on figure 1. This effect produces that the relation between ductility, defined as before, and inelastic capacity of the structure is modified.

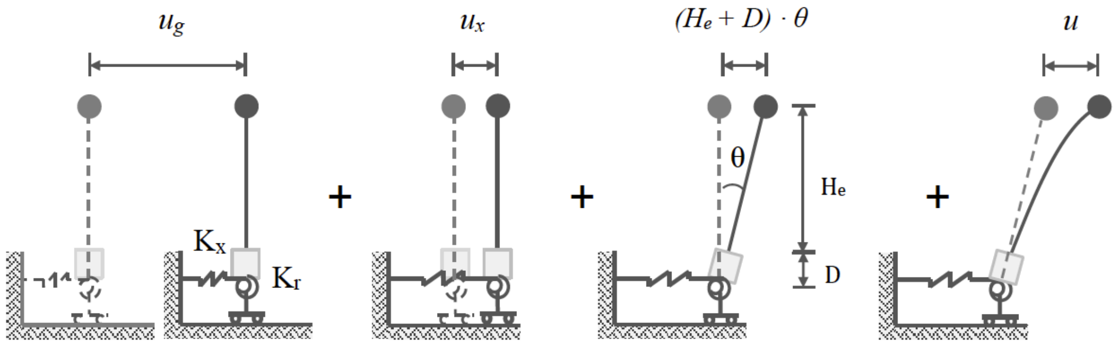

Fig 1- Displacement components of the structure with flexible base.

Previous studies had explored the modifications introduced by DSSI on the inelastic behavior of the structures. Some researches characterize the modifications introduced by flexible base using the concept of an equivalent system with a single degree of freedom (ESDOF) [2-5]. They establish the equivalent properties of a single degree of freedom system (fundamental period, damping ratio and ductility) that may reproduce the inelastic response of a multi degree system with flexible base. This approach is the one used in several building codes to take into account for the DSSI [6-9]. Equivalent ductility ($\tilde{\mu}$) is computed as a function of the fundamental period and ductility of the system with fixed base (T and μ) and the equivalent period ($\tilde{T}$) with Eq. 1. They consider that the ESDOF behaves as a perfect elastoplastic system, with no post yield stiffness. In redundant systems, where many elements contribute to the lateral stiffness, capacity curves show a progressive yield, that must be modeled as a bilinear system with a post yield stiffness. The contribution of the displacement components due to rigid body behavior on the inelastic branch will be smaller, but not null, as for an elastoplastic model.

$$\tilde{\mu} = \left(\frac{T}{\tilde{T}}\right)^2 (\mu - 1) + 1 \tag{1}$$

Equivalent ductility always yields to smaller values than the fixed base ones. This does not mean that the structure with flexible base has a reduced inelastic capacity, a concept that is commonly misunderstood. The correction on ductility factor must be performed due to the modification in the relation between yield strength reduction factor (R_μ) and ductility factor (μ) produced by base flexibility. If elastic forces are reduced by the fixed base yield strength reduction factor without any correction, ductility demands on the structure with flexible base may be increased. This effect will be discussed in the following sections.

ESDOF approach is very useful and yields to good results in a lot of cases. Since just one degree of freedom is used, this procedure considers that modifications introduced by base flexibility in all structural responses along the structure will be linearly equivalent. However, there are studies that have shown that, in some cases, the representation of a flexible base system with multiple degrees of freedom with an ESDOF may not yield in to good results.

Barcena and Esteva [10] studied the ductility demands on multistory systems with flexible base. They have shown that the modification of the ductility demands produced by DSSI are different along the structure height. This effect can not be represented by an ESDOF. Ganjavi and Hao [11] compared the modification on the global ductility demand of structures modeled with flexible base considering multiple degrees of freedom and with the ESDOF approach. Results prove that for very flexible structures, ductility demands with flexible base computed with the ESDOF approach are smaller than the ones computed with the multiple degrees of freedom systems. On the other hand, Fernandez-Sola et al. [12] studied the capacity curves of steel braced frames with flexible base. They conclude that in general, the inelastic capacity of the systems with fixed and flexible base are very similar.

Other modification that flexible base could introduce on the inelastic behavior of structural systems is the variation of the P-Δ effects, since base flexibility increases, in general, the lateral displacements of the structure. Inelastic capacity of the structure and collapse mechanism can be affected by this effect. Hermenegildo and Fernandez-Sola [13] presented an analysis of the influence of base flexibility on the inelastic behavior of steel columns subjected to lateral loads. It is shown that base flexibility reduces booth, yield shear and inelastic capacity of the columns due to the increment of the P-Δ effects.

This work explores the inelastic behavior of a 10 story regular building with RC frames with fixed and flexible base. Non linear static analysis (pushover) is performed in order to stablish the inelastic capacity of the structures. Ductility capacity is computed from the capacity curves. These curves are computed in two different ways. First, the total displacement of the soil-structure system is considered, including the rigid body components (u_x and θ). This set of results are computed to compare the inelastic parameters of the multistory building with the equivalent properties proposed by the ESDOF approach. On the other hand, capacity curves are computed using only the displacement associated with the structure deformation (u), to establish if the inelastic parameters of the structure are modified by base flexibility due to P-Δ effects.

In addition, the non linear dynamic behavior of the buildings are analyzed. The modifications of the hysteretic loops of the whole building and the global ductility demands are studied. The hysteretic loops of an inner and an outer column are compared too.

2. Building and foundation characteristics

Building was designed following the procedure described on the Mexico City Building Code (MCBC) [6]. Design and elements dimensions' details can be found on [14]. On figure 2 representative schemes of plain and elevation view of buildings are presented. RC moment resistant frames are used. Frames are designed with moderate ductility criteria (μ=2) accordingly to MCBC. Fundamental period of the building with fixed base is T=0.83 s. Soil properties correspond to a soft soil represented by a homogeneous layer with thickness of Hs= 40 m and shear wave velocity of Vs= 100 m/s. Foundation consists on a mat foundation overlaying this homogenous soil layer. The foundation is embedded 5 m.

Base flexibility is introduced by using a set of distributed springs along mat foundation (fig. 2). The constants of the springs are computed with the dynamic stiffness concept (impedance function) as presented by Gazetas [15]. This approach considers the influence of the soil mass and stiffness, so the dynamic stiffness of the soil-foundation system depends on the frequency of the excitation. Software DYNA6 [16] was used to estimate horizontal and rocking impedance functions. Since a static and time domain analysis were performed, only the value corresponding to the fundamental frequency of the soil-structure system was used. Given that the period of the soil-structure system with flexible base ($\tilde{T}$) and base flexibility are mutually dependent, it is necessary to perform an iterative process to establish the definitive values of impedance functions. Soil-foundation system stiffness' (K_x and K_r) and periods of the structure with fixed and flexible base are presented on table 1. Additional damping introduced by DSSI is taken into account by using an effective damping ratio. Effective damping ratio was computed with the procedure included on MCBC [6]. Kinematic interaction is neglected.

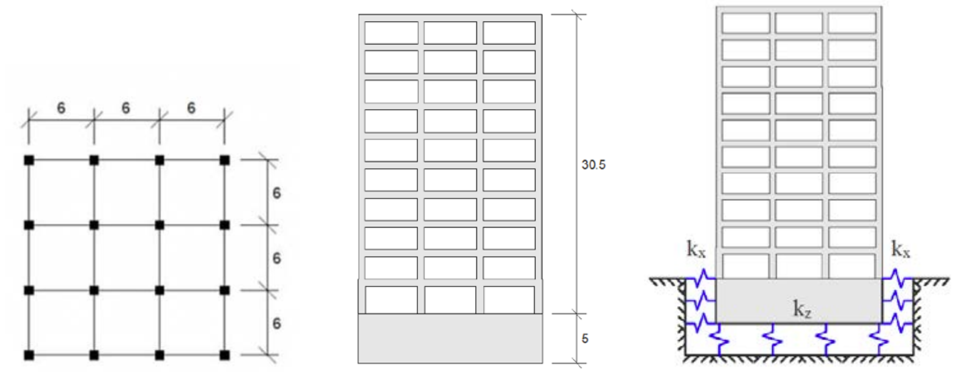

Fig. 2 – Building plain and elevation and base flexibility model.

Table 1. Fixed and flexible base properties of the structure

Soil	$\tilde{T}$ (s)	K_h (t/cm)	K_r $(t \cdot cm)$	ζ_h (%)	ζ_r (%)	$\tilde{\zeta_e}$ (%)
FB	0.83	∞	∞	0	0	5
S2	1.08	3.72x10^2	4.50x10^8	22.1	2.6	4.9

Horizontal stiffness was uniformly distributed among 24 horizontal individual springs (k_x). Rotational stiffness was distributed considering the contribution of the horizontal springs and 16 additional vertical individual springs (k_z). Columns base are constrained with a master joint with a rigid body constrain. More details of this procedure can be found on [16].

Ground motion corresponds to Viveros station record of the 09/19/1985 Mexico City earthquake, with dominant periods around 0.6-0.8 s (figure 3). In order to meet the spectral acceleration values considered on the code, excitation was scaled to meet spectral pseudo accelerations around 1 g for structures fundamental periods with fixed base.

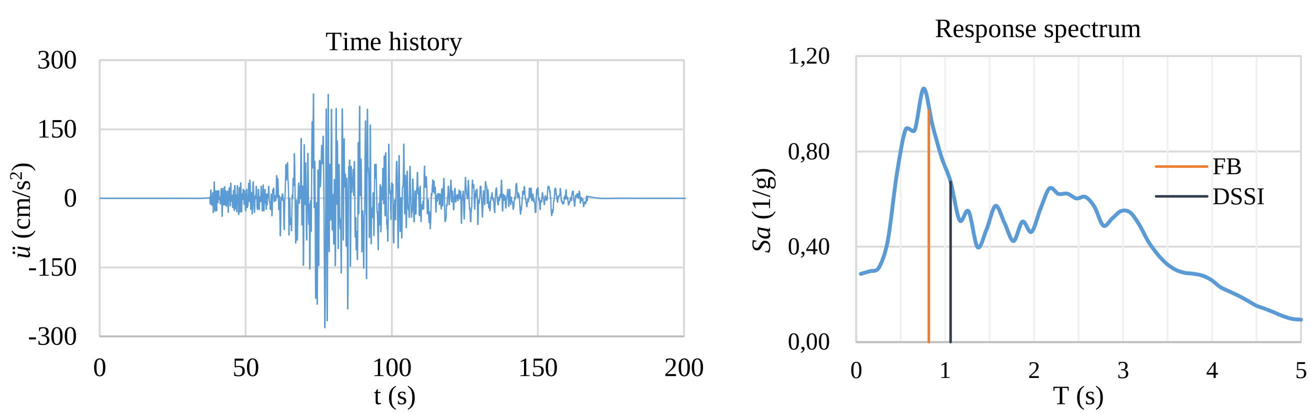

Fig. 3 - 1985 Mexico City earthquake, Viveros station ground acceleration and pseudo acceleration response spectrum.

3. Numerical analysis and results

3.1 Pushover analysis

A triangular load pattern is used for the pushover analysis with displacement control. No-linear behavior of elements (beams and columns) is described by the definition of the nonlinear moment-curvature relations. For the columns, the influence of axial force on the non linear behavior is considered. Table 2 reports the nonlinear parameters for the sections considered. Inelastic parameters are: yield moment (M_y), maximum moment (M_u), curvature ductility (μ_φ), plastic length (l_p) and plastic rotation (θ_p). Possibility of plastic hinges are defined at 5 and 95 percent of the length for frame elements. Plastic length is computed with the empiric equation proposed by Park and Paulay [17].

Table 2- Non linear parameters of frame sections (beams and columns).

	Story	Dimension (cm)	$M_y\ (ton-m)$	$M_u\ (ton-m)$	$\mu_\varphi = \frac{\varphi_u}{\varphi_y}$	$l_p(cm)$	$\theta_p\ (rad)$
Beams	1	50x70	81.85	87.52	7.35	40	0.01030
	2-6	45x70	90.58	97.79	6.66	40	0.00941
	7-10	40x70	63.25	65.76	9.71	40	0.01332
Columns	1	80x80	162.17	200.41	5.53	45	0.00724
	2-7	70x70	136.01	168.46	4.40	40	0.00594
	8-10	65x65	92.12	112.66	4.92	35	0.00625

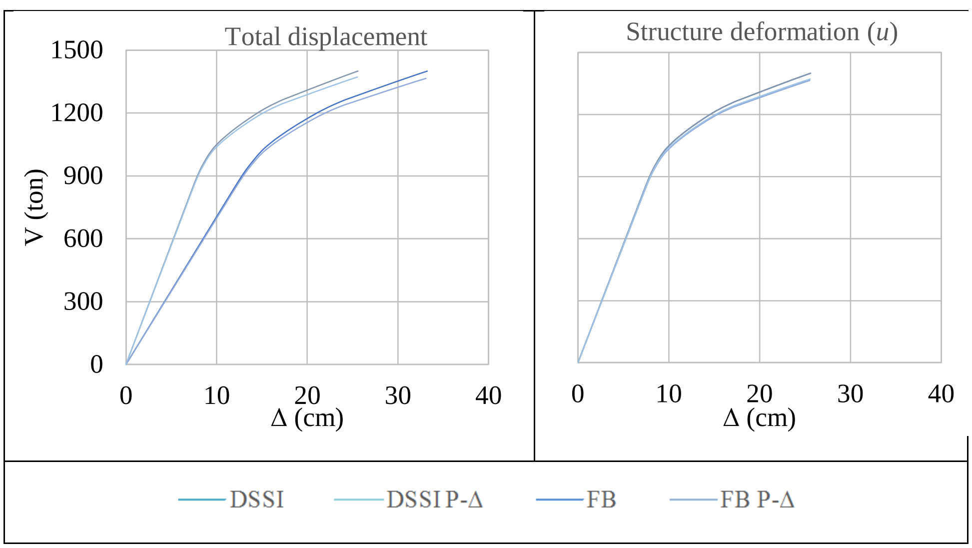

Fig. 4 – Capacity curves of the building with fixed (FB) and flexible base (DSSI) with and without P-Δ effects considering total displacement (left) and structure deformation (right).

Capacity curves (base shear-average drift) of frames are presented in figure 4. The average drift is computed as the ratio of the displacement of the top of the building and building total height. Final mechanism is defined when one of the following three conditions is achieved: a) a plastic hinge develops a rotation greater than the maximum rotation feasible for that element; b) all columns of the same story develop plastic hinges at both ends, producing a soft story failure mechanism and c) all element ends that concur at one joint develop plastic hinges, producing a joint plastic mechanism.

As mentioned before, to asses the ductility variations on buildings with fixed and flexible base, two sets of results are presented. First, capacity curves are computed considering total displacement which includes both the displacement associated with structure deformation (u) and the displacement produced by rigid body behavior (u_x and θ). Ductility reductions computed with this set of results can be associated with the increment of the yield displacement. The second set of capacity curves are computed using only the displacement associated with structure deformation (u). Capacity curves for the building with fixed and flexible base are presented in figure 4 with and without P-Δ effects.

Using the capacity curves, idealized primary curves were constructed to define yield and maximum shear and displacement. With yield and maximum values, the ductility factor is defined. On table 3, ductility factor computed for the structure with fixed (FB) and flexible base (DSSI) are shown. It can be seen that when total displacement is considered, base flexibility reduces the developed ductility (μ). This ductility reduction could be associated with the increment of yield displacement as described in [2-5]. On the other hand, when only the displacement associated with structure deformation is considered (μ^{st}) ductility remains constant. This is an expected result since the inelastic capacity of the building must be independent on the base condition if P-Δ effects are small enough to not change structures behavior [12]. It can be seen from figure 4, that capacity curves with and without P-Δ effects are very similar. In addition, effective ductility ($\tilde{\mu}$) computed with equation 1 is shown. It can be seen that values of ductility computed with the ESDOF approach are very similar to the ones computed with total displacement for this case. This result is in good agreement to the study presented by Ganjavi and Hao [11]. According with that study, for the parameters considered in the present paper ($T = 0.83$ s, ductility of $\mu = 2$ and dimensionless frequency $a_0 = 2\pi H_1/T\,V_s = 2$) ESDOF represent correctly the system with multiple degrees of freedom. For the studied building, effective height for the fundamental mode can be approximated as $H_1 = 0.7\ H_e = 21.35$ m [6].

The main reason of having a reduced value on ductility computed with flexible base, is the modification of the contribution of rigid body components to the total displacement when the structure undergoes inelastic behavior. This effect is produced because the displacement due to rigid body components is not affected by the inelastic behavior of the structure. Consider the common definition of ductility as in equation 2:

$$u_u = \mu u_y \tag{2}$$

where:

$u_u = u_{rb}^u + u^u$	maximum displacement of the soil-structure system
$u_{rb}^u = u_x^u + \theta^u(H_e + D)$	displacement components due to rigid body at maximum displacement
u^u	structure deformation at maximum displacement
$u_y = u_{rb}^y + u^y$	yield displacement of the soil-structure system
$u_{rb}^y = u_x^y + \theta^y(H_e + D)$	displacement components due to rigid body at yield displacement
u^y	structure deformation at yield displacement

Expressing the system ductility in terms of rigid body displacement and structure deformation yield to (Eq. 3)

$$u_{rb}^u + u^u = \mu(u_{rb}^y + u^y) \rightarrow u^u\left(1 + \frac{u_{rb}^u}{u^u}\right) = \mu u^y\left(1 + \frac{u_{rb}^y}{u^y}\right) \tag{3}$$

As mentioned before, to compute the ductility in the structure (μ^{st}), only the displacement produced by structure deformation must be considered. In consequence, the relation between the overall system ductility with flexible base (μ) and ductility in the structure (μ^{st}) can be defined as (Eq. 4)

$$\mu^{st} = \frac{u^u}{u^y} = \mu \frac{\left(1 + \frac{u_{rb}^y}{u^y}\right)}{\left(1 + \frac{u_{rb}^u}{u^u}\right)} \tag{4}$$

If the the contribution of rigid body components to the total displacement remains constant for booth yield and maximum displacement, ductility on the structure is equal to the ductility on the system. However, if the contribution is modified, the ratio between μ and μ^{st} will change.

Table 3 shows the percentage of contribution of the different components to the total displacement at yield (u_y) and maximum (u_u) displacements. It can be seen that for u_y, rigid body components (u_{rb}^y) represent 38% of the total displacement, while for u_u the contribution of these components (u_{rb}^u) to the total displacement is reduced to a 23%. With these values, equation 4 yields to:

$$\mu^{st} = \mu \frac{\left(1 + \frac{0.38}{0.62}\right)}{\left(1 + \frac{0.23}{0.77}\right)} = 1.24\mu \rightarrow \mu = \frac{\mu^{st}}{1.24} = \frac{2.67}{1.24} = 2.15$$

This procedure to estimate system ductility fits better the values obtained from the capacity curve in comparison with the one computed with the ESDOF approach (table 2). The difference is that the ESDOF approach does not take into account the post yield stiffness of the system.

Table 3- System (μ), structure (μ^{st}) and equivalent ($\tilde{\mu}$) ductility and displacement components for yield and maximum displacements (u_y and u_u).

	Ductility capacity			Displacement components at u_y (%)			Displacement components at u_u (%)		
System	μ	μ^{st}	$\tilde{\mu}$	u_x^y	$\theta^y(H_e + D)$	u^y	u_x^u	$\theta^u(H_e + D)$	u^u
FB	2.67	2.67	2.67	-	-	100.00	-	-	100.00
DSSI	2.12	2.67	2.00	2.17	35.83	62.00	1.31	21.69	77.00

3.2 Non linear time history analysis

Ductility demands were obtained by non linear time history analysis. Global and local ductility demands and hysteretic loops were computed. For global behavior, as well as in the pushover analysis, two types of results were computed for the building with flexible base, one considering the total displacement and other considering only the structure deformation. For the analysis of element ductility and hysteretic loops, only the element deformation is considered.

Non linear parameters for elements (beams and columns) are the same as for the pushover analysis. Stiffness degradation is taken into account considering the hysteretic model proposed by Takeda [18]. Analysis is performed for the whole duration of the excitation, since none of the elements achieves its maximum plastic rotation in any moment. Only inelastic behavior for flexure is considered, since design procedure considers that shear failure is avoided.

Figure 5 shows the global base shear-displacement curves, for fixed and flexible base, considering the roof displacement. Results with and without P-Δ effects are shown. It is clear that responses with and without second order effects are very similar for this case. It can be seen that, as expected, the displacements of the system with flexible base are larger due to increased flexibility in the same way that for pushover analysis. In this case, maximum displacement with fixed base is 13.60 cm while with flexible base is 23.18 cm, with an increment on displacement of approximately 70%. Maximum base shear is slightly reduced due to base flexibility (1,206 ton for fixed base and 1,182 ton for flexible base). This reduction is associated with the reduction on the spectral acceleration produced by the increase in period as shown on figure 3. Another modification is on the symmetry of the response. For fixed base, the ratio of the maximum positive and negative displacement is 0.92 while for flexible base is 0.87, making the response with flexible base slightly less symmetric.

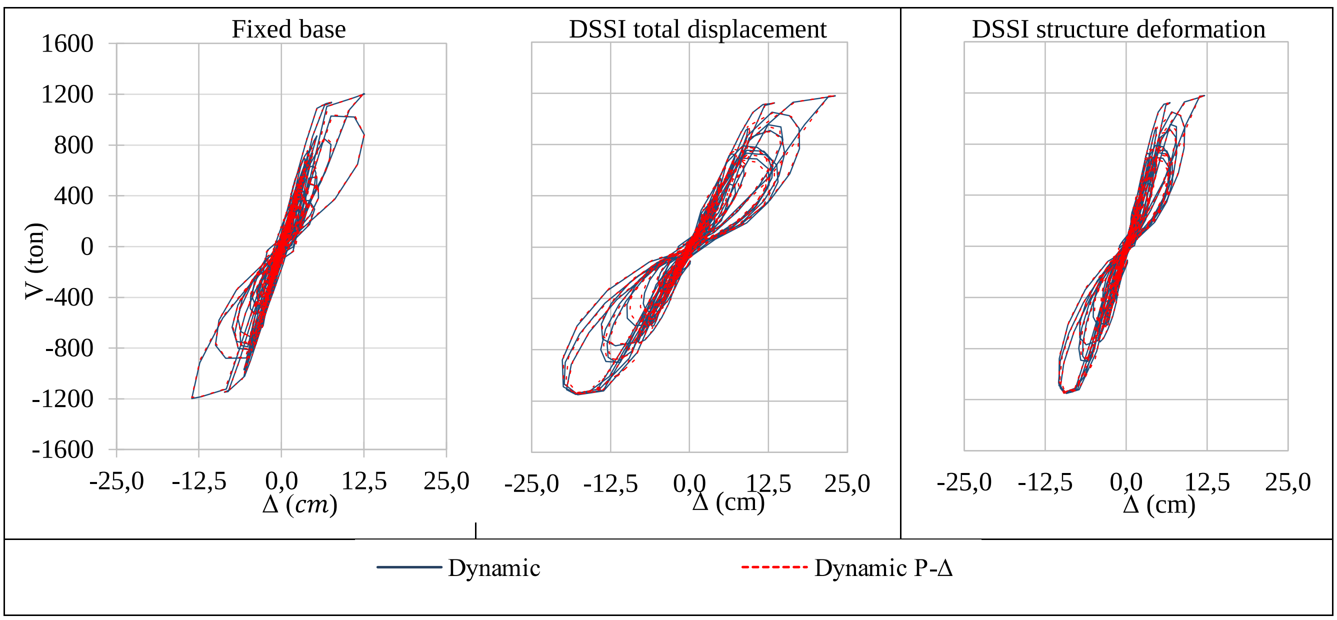

Fig. 5- Base shear – roof displacement curves for the non linear time history analysis of the building with fixed base, flexible base considering total displacement and structure deformation.

Ductility demand is modified too. Global ductility demand goes from μ_{FB}=2.64 for the fixed base system to μ_{DSSI}=2.26 for the system with flexible base. In this case, the modification on the ductility demand is associated with the different yield strength reduction factor of booth systems (R_{μ}). Since the building strength remains constant, regardless base stiffness, yield base shear is the same for booth cases (V_y=1,100 ton). However, base shear associated with the elastic spectrum are different due to the modification on the fundamental period. For the fixed base system, elastic base shear is V_{0FB}=3,013.20 ton, yielding to a $R_{\mu FB}$=2.74. For the flexible base system, elastic base shear is V_{0DSSI}=2,138.40 ton and $R_{\mu DSSI}$=1.94

As mentioned before, for flexible base systems the ductility demand in the structure must be computed subtracting the rigid body components. Figure 5 shows the response of the flexible base system considering only the structure deformation. Since elastic base shear and yield base shear are the same without rigid body components, value of $R_{\mu DSSI}$=1.94 is not modified. On the other hand, ductility demand on the structure must be computed eliminating rigid body components, yielding to a value of μ^{st}=2.36, that is larger than the demand over the whole soil-structure system. As for the pushover, the difference is produced due to the modification on the contribution of the rigid body components. On the dynamic analysis, rigid body components contribute with 50.4% to the total yield displacement and with 47.5% to the maximum displacement. The ratio between ductility demand of the complete soil-structure system (μ_{DSSI}) and the ductility demand of the structure (μ^{st}) can be computed with Eq. 4 considering the contribution of rigid body components. Eq. 4 yields to a value with a good agreement with the demand computed from the non linear analysis (μ^{st}=2.36).

$$\mu^{st} = \mu_{DSSI}\frac{\left(1+\frac{0.504}{0.496}\right)}{\left(1+\frac{0.475}{0.525}\right)} = (2.26)(1.059) = 2.39$$

The design procedure of inelastic structures is based on the estimation of specific values of R_{μ} that control ductility demand on the structure to a target value. Since μ^{st} is always larger than μ_{DSSI}, it is necessary to use reduced $R_{\mu DSSI}$ values to keep μ^{st} within design values. For example, for the studied structure if the structural system has a ductility capacity of μ^{st}=2.26, it can be misunderstood that a value of $R_{\mu DSSI}$=1.94 can be used, since a ductility demand on the soil-structure system of μ_{DSSI}=2.26 is produced. However this reduction factor will produce a ductility demand on the structure of μ^{st}=2.36, exceeding the capacity. If a target structure ductility demand of μ^{st}=2.26 is desired, a reduced value of $R_{\mu DSSI}$ must be considered. For this reason, some building codes considers reduced values of inelastic capacity of systems with flexible base for the computation of yield strength reduction factors [6]. Reduced inelastic capacity is computed with Eq. 1. For the studied system, if a target structure ductility of μ^{st}=2.36 is desired, a modified global ductility of the soil-structure system (μ_{DSSI}) must be used for computing $R_{\mu DSSI}$. Value of μ_{DSSI} can be computed with equation 1 and equation 4.

$$\mu_{DSSI} = \frac{(0.83)^2}{(1.08)^2}(2.36-1)+1 = 1.80$$

$$\mu_{DSSI} = (2.36)\frac{\left(1+\frac{0.475}{0.525}\right)}{\left(1+\frac{0.504}{0.496}\right)} = 2.23$$

It is clear that equation 1 yields to smaller values of global ductility in comparison with equation 4, and the differences between booth equations are larger for this case than for the pushover analysis. As mentioned before, equation 1 considers elastoplastic behavior of the system while equation 4 takes into account the actual contribution of rigid body components. For design purposes, equation 1 is more suitable since the only parameters required are the target ductility demand on the structure and the fixed and flexible base fundamental periods.

A fundamental parameter on designing inelastic structures is the ratio between yield strength reduction factor and ductility demand on the structure (R_{μ}/μ). It is clear that this ratio is different for booth cases. For fixed base this ratio yields to $R_{\mu FB}/\mu_{FB} = 1.04$ while for flexible base yields to $R_{\mu DSSI}/\mu^{st} = 0.82$. For the fixed base case, it can be seen that ductility demand is almost equal to the reduction in the strength. However, for flexible base, despite the system fundamental period is larger, ductility demands are larger than the reduction in the strength.

For the analysis of individual elements, hysteretic loops for two ground floor columns (an outer and an inner column) are shown on figure 6. The relation of flexural moment and plastic rotation are shown. Rotational maximum ductility demands in the fixed base structure are μ_{FB}=1.79 and 2.01 for outer and inner column respectively. On the other hand, maximum demands on the flexible base outer and inner columns are $\mu_{\theta DSSI}$=1.43 and 1.62. Figure 6 shows that hysteretic loops for columns in the fixed base structure are non-symetric, having larger positive plastic rotations. In the building with flexible base, hysteretic loops are smaller and more symmetric. For this case, modifications of ductility demands on booth columns due to base flexibility are very similar. However, the ratio of global displacement ductility demands and local rotational ductility demands are modified. On table 4, these ratios are shown for booth columns.

Table 4. Global to local ductility ratio for two ground floor columns (inner and outer) on the building with fixed and flexible base

Column	$\mu_{FB}/\mu_{\theta FB}$	$\mu_{DSSI}/\mu_{\theta DSSI}$
Inner	1.45	1.65
Outer	1.31	1.46

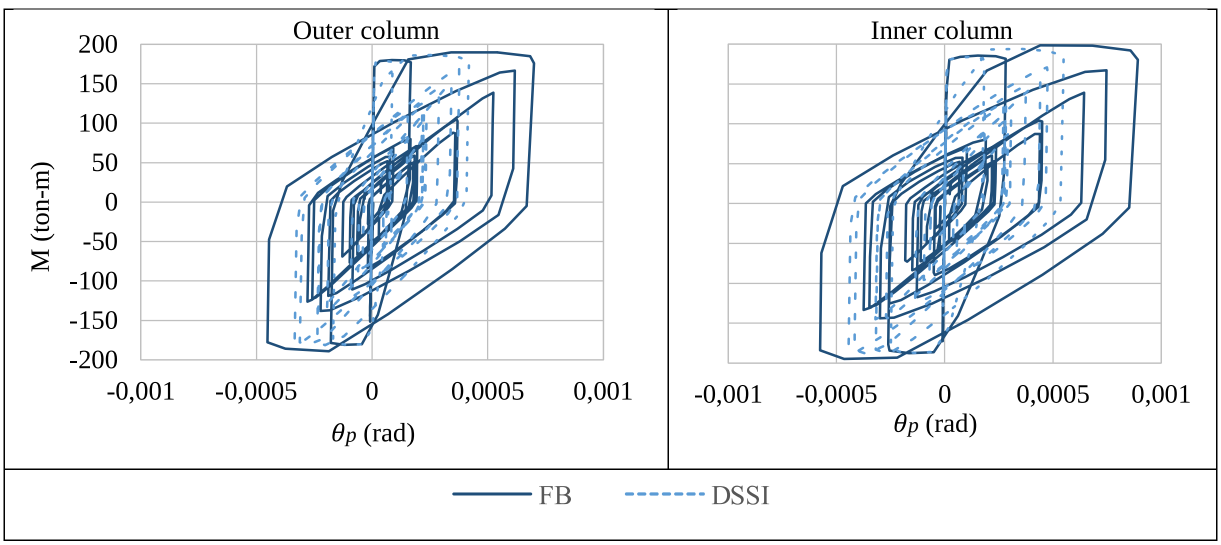

Fig. 6- Hysteretic loops of ground floor columns for the structure with fixed and flexible base.

4. Conclusions

Inelastic static and time history analysis of RC frames with fixed and flexible base are presented. A moment resistant building with 10 stories is considered with a mat foundation. Ductility capacities are defined based on the idealized base shear-average drift capacity curves from the static non-linear analysis. Ductility demands and yield strength reduction factors are computed from time history non-linear analysis. Average drift was computed in two ways: one with the total displacement of the soil-structure system, which includes structure deformation and rigid body components and other considering only the structure deformation. Impedance functions for the fundamental frequency is used.

From the static non-linear analysis, it is shown that when total displacement is considered (whole soil-structure system), ductility is reduced by base flexibility in general. Ductility reduction in this case is mostly due to the increment on yield displacement produced by system flexibilization. Ductility reduction does not mean a reduction on deformation capacity, it is produced by the difference on the contribution of rigid body components to total displacement at yield and maximum displacement. When only the displacement associated with structural deformation is used, ductility changes are almost null. It means that inelastic capacity of the structure remains equal independently on base flexibility. An expression to compute the relation of soil-structure system ductility and structure ductility is proposed. This approach considers explicitly how the the contribution of rigid body components influences this relation.

Global ductility demand and the corresponding yield strength reduction factor are modified by base flexibility due to the change of fundamental period. For the flexible base structure, ductility demand on the whole soil-structure system is smaller than the actual ductility demand produced on the structure. It is proved that the ratio between yield strength reduction factor and structure ductility demand is smaller for the structure with flexible base respect to the fixed base case. For local ductility, it is shown that columns on the building with flexible base develop smaller ductility demands and smaller hysteretic loops.

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