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Strong-Back System Coupled with Framed Structure to Control the Building Seismic Response

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Abstract

In the present paper, the coupled behavior of structural systems obtained by connecting a moment resisting frame structure with a vertical elastic truss, known in the literature as strong-back, which acts as a mast by imposing to the structure a given lateral deformed shape, is investigated. The rigid behavior of the strong-back, which is designed in order to remain in the elastic field under strong seismic ground motion, imposes a uniform inter storey drift along the frame height, thus avoiding undesired effects such as soft storey and weak storey mechanisms.

Consequently, the whole structural system may be, at first approximation, modelled as an equivalent Single Degree of Freedom system thus allowing for an analytical description of its response. In particular, in the work the attention is paid to the mutual actions exchanged by the strong-back and the frame by solving the static equilibrium equations, assuming a shear type behavior for the frame. Finally, some numerical simulations of frame systems with strong-back systems as subjected to earthquake ground motions are developed, encompassing both shear type frames and frames with flexible beams.

Keywords: Strong-back; Earthquake resistant design; Static behavior; Mutual actions; Numerical simulations

Introduction

New approaches in earthquake resistant design have determined a change of paradigm from the traditional Force Based Seismic Design approach (FBD) to the so-called Performance Based Seismic Design (PBSD) approach based on the imposition of specific performance objectives (POs), obtained by considering both the structural and non-structural performance of the building under increasing seismic intensity levels [1]. The basic idea lies upon the capacity of predicting that a given system will perform in a selected manner (i.e., performance level) under a given seismic intensity (i.e., earthquake design level) [2,3].

First of all, the desired performance design objectives need to be expressed in terms of precise engineering demand parameters (EDPs) capable of measuring various and different kinds of damages, resulting from expected levels of earthquake ground motions. Then by imposing specific values of EDPs it is possible to move ranging from code requirements to superior performances such as fully operational level guaranteed under very rare earthquake. Inter-storey drifts proved to be a reliable parameter to quantify damages in framed structures [4].

In view of this, a possible strategy for superior seismic performances under earthquake excitation should be based on the use of peculiar solutions for limiting the attitude of conventional frames in developing large drifts concentrated in one or few stories, which often lead to undesirable seismic responses such as soft storey mechanisms, thus leading to larger non-structural and structural damages and even premature collapses (when compared to the structural systems allowing for a more uniform distribution of damage over the height). Moreover, soft storey mechanisms are also likely to result in significant residual displacements, which can be extremely costly or even unfeasible to repair.

One possible solution for an enhanced seismic performance of frame structures is the use of adhoc designed hysteretic steel braces, such as those developed by Christopoulos and coworkers at the University of Buffalo (commercially known as Scorpion brace devices) [5] or the ones developed by some of the authors of the present work and known as Crescent Shaped Brace [6-8].

Alternatively, a novel hybrid system composed of a traditional frame and a mast, known as strong-back system, has been recently proposed by researchers at University of California Berkeley to achieve improved seismic performances [9]. The mast, in fact, acts like a “strong back”, to help resist the tendency of frames to concentrate damage in one or few stories during severe seismic excitations. The mentioned studies were mainly devoted to the sizing of specific trussed systems so that the steel members would remain in their elastic field under severe ground motions, thus ensuring the development of nearly uniform inter-storey drifts along the building height. Despite those studies, no further work was carried out to investigate the coupled nature of the system response under lateral loads and to evaluate the mutual actions exchanged between the frame and the strong-back. Indeed, even though a lot of research work was focused on the lateral response of coupled structural systems, the available results refer to the interactions between shear wall and frame system, composed by a frametype structure connected to a walltype structure [10-15]. To this purpose, the seminal works done by Khan [10], Rosman [11] and Stafford Smith [12] back to 1960s and 1970s, devoted to the comprehension of the mutual interactions between the frame and the shear wall through analytical approaches, still nowadays represent the fundamental body of knowledge for the comprehension of the coupled response of such complex systems.

By using a similar approach, in the present work the aim is to obtain analytical expressions of the mutual actions exerted by the frame and the strong-back to fully understand the behavior of these structural systems.

Strong-Back System: The Concept and Previous Studies

The so-called strong-back system, as schematically represented in Figure 1, has been first introduced by Lai and Mahin [9]. The system is essentially a vertical truss going from the top to the ground storey made

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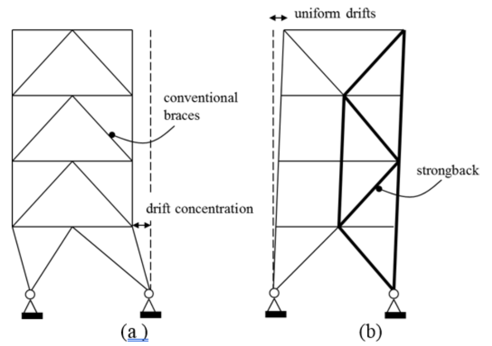

Figure 1: (a) Conventional frame (b) Frame with strong-back system (c) Mutual forces exerted between the frame and the strong-back [9].

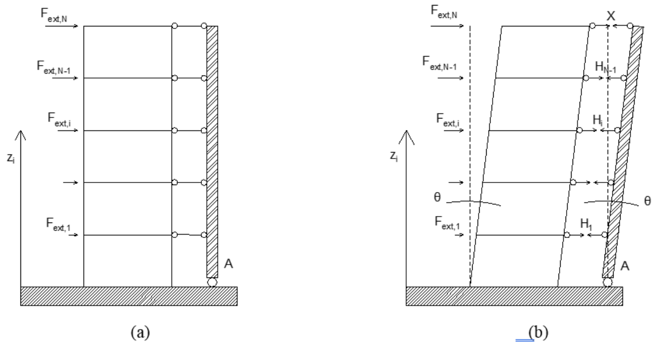

Figure 2: (a) Schematization of the coupled frame strong-back system (b) External horizontal forces and mutual actions applied to the hybrid system.

by segments of the augmented braced bay which are proportioned to provide a continuous vertical truss that remains essentially elastic during strong ground motions. Its main function is to avoid large deformations which can be concentrated at particular weak stories during an earthquake. Indeed, the vertical truss provides an elastic "strong back" or mast, which imposes a nearly uniform lateral deformed shape over the height of the structure (Figure 1).

The original idea was inspired by previous research studies aimed at reducing the damage concentration and achieving smaller residual displacements by connecting the main frame to an additional system: (i) dual systems, where a moment resisting frame is used in addition to a braced frame [16,17] (ii) zipper or vertical tie bar systems [18,19] (iii) rocking/uplifting systems [20-22] (iv) tied-truss, masted systems [23,24].

The work previously done focuses on the investigation of the novel hybrid system performances through non-linear inelastic analysis results of a total of 6 different prototypes of braced frame systems, 3 of which equipped with strong-back systems, under a variety of earthquake excitations. For this purpose, two-dimensional computer models were developed in OpenSees, in which static and cyclic pushover analyses as well as nonlinear dynamic response history analyses were performed. The results of the study show that the strong-back system prevents the deformation concentration in steel braced frames, thus avoiding soft storey mechanism and additional cost comparison evidences that the new hybrid system would be economically feasible with respect to the traditional braced frames.

In the aforementioned research conducted by Lai and Mahin [9], however, the strong-back system is idealized as a braced frame in which part of the bay is augmented so to be modeled as a vertical truss that remains essentially elastic under seismic action, with no evidence of analytical studies focused on the interaction between the "strong back" and the frame system.

In the present study the strong-back concept is adapted and considered as an external stiff system with linear elastic behavior even under high levels of lateral load, and the interaction between the strong-back and the frame, which is essential to capture the real behavior of the hybrid system, is investigated following the approach of Khan [10], Rosman [11] and Stafford Smith [12] used to comprehend the behavior of shear wall frame systems.

Behavior Under Equivalent Static Lateral Loads

The system schematization

Let us consider the system schematization as in Figure 2, representing a generic *N*th storey frame system coupled with a strong-back system, pinned at the base. At the *i*th storey the frame is characterized by a lateral stiffness equal to k_i and a floor mass equal to m_i. The geometrical configuration of the system can be described by assuming a system coordinate x, z having origin at the base of the strong-back (point A).

The structure when subjected to the generic set of external lateral forces $F_{ext,i}$ develops a lateral displacement profile d_i, as well as a set of internal mutual horizontal forces H_i exchanged between the frame and the strong-back system (Figure 2b). Hereafter, the mutual action at the top storey is going to be referred to as *X*.

Under the following additional assumptions:

• Both the frame and the strong-back system have linear elastic behavior.

• The beams are considered as infinitely stiff (shear type schematization).

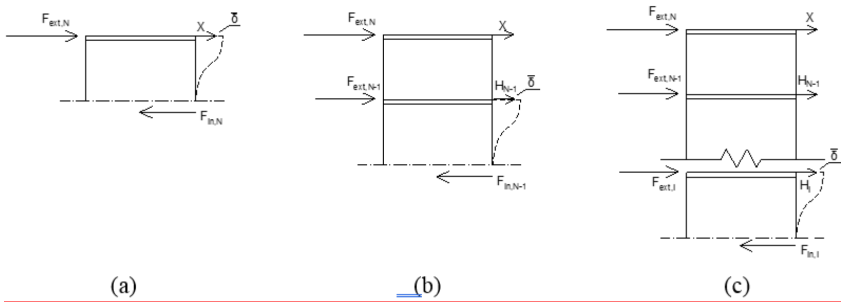

Figure 3: Schematization of the system cutting: (a) at the Nth storey (b) at the N[th]1 storey (c) at the i[th] storey.

- The axial flexibility of columns is neglected.
- The strong-back is infinitely rigid.

The lateral drift profile is linear and therefore it can be completely described in terms of the unique rigid rotation ρ

Assuming equal storey height at all stories (e.g. $h_i = h \ \forall i$) it follows that the inter storey drifts ($\delta_i = d_i - d_{i-1}$) has to be constant along the entire building height (e.g. $\delta_i = \overline{\delta} \ \forall i$).

Equilibrium equations

Let us consider the translational equilibrium equation written by cutting the system just below the *N*th floor (Figure 3a). External force $F_{ext,N}$ has to be in equilibrium with the internal mutual force *X* and the internal storey shear $F_{in,N}$,so that:

$$F_{ext,N} + X = F_{in,N} \tag{1}$$

where $F_{in,N} = k_N \delta_N = k_N \overline{\delta}$.

Eq. 1 allows to obtain the following expression of the constant interstorey drift $\overline{\delta}$:

$$\overline{\delta} = \frac{F_{ext,N} + X}{k_N} = \frac{F_{ext,N}}{k_N} + \frac{X}{k_N} \tag{2}$$

Let us consider the translational equilibrium equation written by cutting the system just below the *N*th floor (Figure 3b). Again the sum of the external forces has to be in equilibrium with the sum of internal mutual forces exchanged between the frame and the strong-back, and the internal storey shear, so that:

$$F_{ext,N} + F_{ext,N-1} + X + H_{N-1} = F_{in,N-1} \tag{3}$$

where $F_{in,N-1} = k_{N-1}\overline{\delta}$.

By combining Eq. 2 and 3 the mutual action H_{N1} can be expressed in terms of the external forces and the top mutual action:

$$H_{N-1} = A_{N-1} \cdot F_{ext,N} + A_{N-1} \cdot X - F_{ext,N-1} \tag{4}$$

Where $A_{N-1} = \frac{k_{N-1} - k_N}{k_N}$ is a coefficient depending upon the relative differences of the lateral storey stiffness of the two top stories?

Similarly, by cutting the system at the generic *i*th storey, Figure 3c the translational equilibrium equation can be written as follows:

$$F_{ext,N} + \sum_{j=i+1}^{N-1} F_{ext,j} + F_{ext,i} + X + \sum_{j=i+1}^{N-1} H_j + H_i = F_{in,i} \tag{5}$$

Where, $F_{in,i} = k_i \overline{\delta}$.

By substituting in Eq. 5 the expressions of the constant inter-storey drift *H* of Eq. 2, the following analytical expression of the mutual force at the generic *i*th storey H_i can be derived:

$$H_i = A_i \cdot X + A_i \cdot F_{ext,N} - F_{ext,i} \tag{6}$$

Where $A_i = \frac{k_i - k_{i+1}}{k_N}$ is a numerical coefficient depending upon the difference between the lateral stiffness of the *i*th storey and the one of the storey above (normalized with respect to the top storey lateral stiffness). It can be noted that H_i can be expressed uniquely in terms of the external force $F_{ext,N}$ and the mutual force *X* at the top floor (both multiplied by the numerical coefficient A_i) and the external force at the *i*th storey $F_{ext,i}$. As such, if the set of the mutual forces is known and the lateral stiffness distribution k_i is also known, the unknown mutual force *X* can be derived by imposing the rotational equilibrium around point A:

$$\sum_{j=1}^{N-1} H_j z_j + X z_N = 0 \tag{7}$$

After combining Eq. 6 and 7 the following analytical expression of *X* can be derived:

$$X = -\frac{F_{ext,N} \sum_{j=1}^{N-1} A_j z_j - \sum_{j=1}^{N-1} F_{ext,j} z_j}{\left(z_N + \sum_{j=1}^{N-1} A_j z_j \right)} \tag{8}$$

Then, substituting Eq. 8 into Eq. 7 the following explicit expression of the mutual forces H_i can be derived:

$$H_i = -A_i \frac{F_{ext,N} \sum_{j=1}^{N-1} A_j z_j - \sum_{j=1}^{N-1} F_{ext,j} z_j}{\left(z_N + \sum_{j=1}^{N-1} A_j z_j \right)} + A_i F_{ext,N} - F_{ext,i} \tag{9}$$

Mutual Actions and Frame Lateral Stiffness

Let us now focus the attention on the expression of the mutual horizontal forces H_i as given by Eq. 6. The expression is made by the algebraic sum of the following three terms:

Ai. X represents the term proportional to the mutual action at the top storey *X*. The value of the coefficient of proportionality A_i is related to the difference between the lateral stiffness of the *i*th storey and that of the storey above (*i*+1). Indeed, as the storey stiffness increases also the mutual action tends to increase as well.

Ai. $F_{ext,N}$ represents the term proportional to the external action at the top storey $F_{ext,N}$. Again, the coefficient of proportionality is A_i.

$F_{ext,i}$ represents the term proportional to the storey shear at the *i*th storey (with constant coefficient of proportionality equal to 1).

In the next subsections the general expressions of H_i and *X* are specified for interesting cases in order to provide some useful considerations.

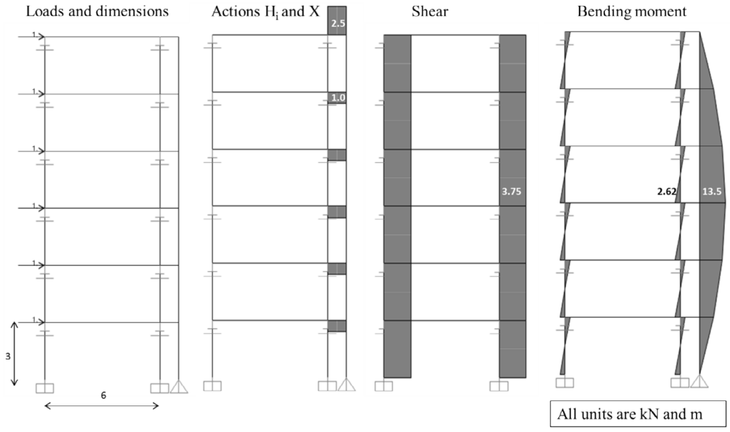

Figure 4: Internal actions in a 6 storey shear type frame with strong-back.

On the lateral stiffness distributions

In the case of a frame structure with uniform lateral stiffness along the height of the structure ($k_i = k\ \forall \mathrm{i}$) the value of coefficient A_i becomes identically null at all stories and the expressions of the mutual actions (Eq. 8 and 9) simplify as follows:

$$X = \frac{M_{ext,A}}{z_N} - F_{ext,N} \tag{10}$$

$$H_i = -F_{ext,i} \tag{11}$$

Where $M_{ext,A} = \sum_{j=1}^{N} F_{ext,j} z_j$ the moment of the external forces around point A is (also called base bending moment). Similarly the summation of the external forces $V_{ext} = \sum_{j=1}^{N} F_{ext,j}$ is also called base shear (especially in seismic engineering).

Eq. 11 simply states that the mutual action at the *i*th storey has equal value and opposite direction with respect to the external force at the same *i*th storey, whereas from Eq. 10 it results that X has an amplitude which is proportional to the base bending moment divided by building height.

It is also of interest to consider the case for which the mutual actions are all identically null. This corresponds to impose the following set of conditions:

$$\begin{cases} H_i = A_i \cdot X + A_i \cdot F_{ext,N} - F_{ext,i} = 0 \\ X = 0 \end{cases} \tag{12}$$

Yielding to the following specific relations between the values of coefficients A_i and the external forces $F_{ext,i}$:

$$A_i = \frac{k_i - k_{i+1}}{k_N} = \frac{F_{ext,i}}{F_{ext,N}} \tag{13}$$

In other words, a frame structure characterized by a lateral stiffness distribution satisfying Eq. 13 when subjected to the given set of static external forces $F_{ext,i}$ will develop a linear storey drift profile.

Solutions for particular external forces profiles

Focusing on the previous case of a frame system with uniform lateral stiffness along the height of the structure ($k_i = k\ \forall \mathrm{i}$), let us now consider the following two specific along the height distributions of external forces.

- **Case 1:** Uniform distribution:

$$F_{ext,i} = F\ \forall \mathrm{i} \tag{14}$$

- **Case 2:** Inverse triangular distribution (typically adopted for seismic analysis):

$$X = F \cdot \left(\frac{N-1}{2}\right) \tag{15}$$

In such special cases the expressions of the mutual forces further specify as follows:

For Case 1:

$$X = F \cdot \left(\frac{N-1}{2}\right) \tag{16}$$

$$H_i = -F \tag{17}$$

Considering that the base shear (e.g. the summation of the external forces) is equal to $V_{ext} = N \cdot F$ the mutual force at the top storey can be also expressed as follows:

$$X = V_{ext} \cdot \frac{N-1}{2N} \tag{18}$$

Note that, for large N, the value of X tends to 0.5 V_{ext}.

For Case 2:

$$X = F \cdot \left(\frac{(N+1)(2N+1)}{6N} - 1\right) \tag{19}$$

$$H_i = -F \cdot \frac{i}{N} \tag{20}$$

Considering that the base shear (e.g. the summation of the external forces) is equal to $V_{base} = F \cdot \left(\frac{N+1}{2}\right)$ the mutual force at the top storey can also be expressed as follows:

$$X = V_{base} \cdot \left[\frac{(N+1)(2N+1) - 6N}{3N(N+1)}\right] \tag{21}$$

Note that for large N, the value of X tends to 0.66 V_{ext}.

For instance, Figure 4 displays the along the height profiles of the internal actions (e.g. mutual actions H_i and X, columns shear and bending moment in the frame, bending moment in the strong-back) for the specific case of a 6storey structure as subjected to a uniform set of unitary external forces (Case 1). It can be noted that the presence of the strong-back leads to a uniform storey shear along the building height, with a constant value (3.75 kN) which is around 50% of the base shear of the same frame but with no strong-back (equal to the sum of the external forces, which is 6 kN). Such a reduction in the base shear is paid by considerably large mutual actions and bending moments in the strong-back, requiring adequate sizing of the connections between

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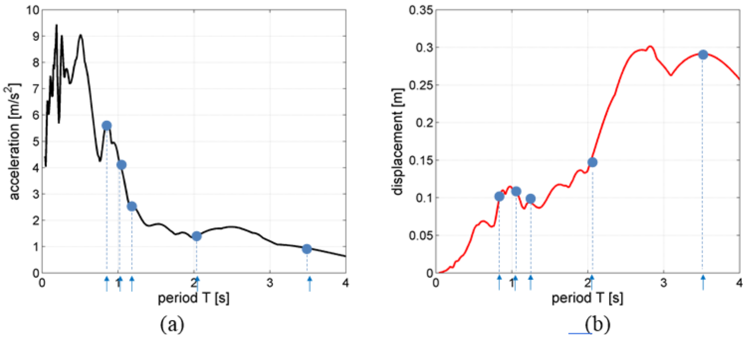


Figure 5: (a) Pseudo acceleration response spectrum and (b) Displacement response spectrum, considering a 5% damping ratio.

ρ \ model	Bare frame	Frame with strong-back system
ρ=0 (PIN)	PIN – BARE (T_1=7.44s)	PIN – SB (T_1=3.51s)
ρ=0.27 (IPE300)	IPE300 – BARE (T_1=2.22s)	IPE300 – SB (T_1=2.04s)
ρ=0.78 (IPE500)	IPE500 – BARE (T_1=1.31s)	IPE500 – SB (T_1=1.25s)
ρ=1.49 (IPE600)	IPE600 – BARE (T_1=1.15s)	IPE600 – SB (T_1=1.10s)
ρ=∞ (ST)	ST– BARE (T_1=0.84s)	ST – SB (T_1=0.80s)

Table 1: FE models developed in the present study.

frame elements and the strong-back, as well as the truss elements forming the strong-back itself.

Let us consider instead the case of inverted triangular profile of external forces (Case 2). In such a case it may be of interest to obtain the stiffness distribution leading to a linear lateral drift profile. Introducing Eq. 15 into Eq. 13, after some simple mathematical manipulations, it is possible to derive the following expression of this particular lateral stiffness distribution:

$$k_i = \frac{k_N}{N}\frac{(N-i+1)\cdot(N+i)}{2} \tag{22}$$

Incidentally, the lateral stiffness distribution of Eq. 22 is also a quite accurate approximation of the lateral stiffness distribution of a *N*storey sheartype frame structure with uniform floor mass distribution and characterized by a linear first mode shape [25].

Numerical Simulations

The FE models and the seismic analysis

It is well known that the distribution of internal actions in a moment resisting frame structure is significantly affected by the beam to column stiffness ratio (Hardy Cross method [26], then extended by Pozzati for the more general case of moment resisting frames subjected to lateral loads [27]:

$$\rho = \frac{E_b J_b}{L_b}\cdot\frac{L_c}{E_c J_c} \tag{23}$$

Where E_b, J_b, L_b are the elastic modulus, modulus of inertia and length of the beam, respectively, while E_c, J_c, L_c are the elastic modulus, modulus of inertia and length of the column, respectively.

In this section, some numerical Finite Element (FE) based simulations are carried out with the purpose of assessing the seismic behavior of moment resisting frames equipped with strong-back systems. In particular, a total number of 10 two bays six storey steel frame structures are investigated, namely 5 bare frame structures and the corresponding 5 counterparts with strong-back systems. All structures have a constant inter storey height *h*= 3 m and a constant span width equal to 6 m. HE300 steel profiles are used for all columns, while the cross sections of the beams are varied in order to obtain beam to column stiffness ratios ρ ranging from 0.25 to 1.5 (in addition to the limited cases of pinned connections and of shear-type structures). In more detail, five different ρ values are used in the numerical simulations:

- ρ=0 (Limit case of pinned connections);
- ρ=0.27 (Relatively flexible beam case obtained using IPE300 beams at all stories);
- ρ=0.78 (Intermediate case obtained using IPE500 beams at all stories);
- ρ=1.49 (Relatively rigid beam obtained using IPE600 beams at all stories);
- ρ=∞ (Limit case of shear-type behavior).

Uniform floor masses are applied at all stories leading to a total seismic weight equal to W_{tot} = 4150 kN. The El Centro 1940 acceleration time history record (North South Component) is used as seismic input at the base of the structure.

Table 1 summarizes the main characteristics (ρ and fundamental periods of vibration for the 10 FE models) while Figure 5 provides the displacement and pseudo acceleration response spectra (considering a 5% damping ratio) of the El Centro 1940 record (the blue dots indicate the spectral ordinates corresponding to the natural periods of the 5 frames with strong-back systems). The numerical simulations are carried out using the commercial software SAP2000 v.18.

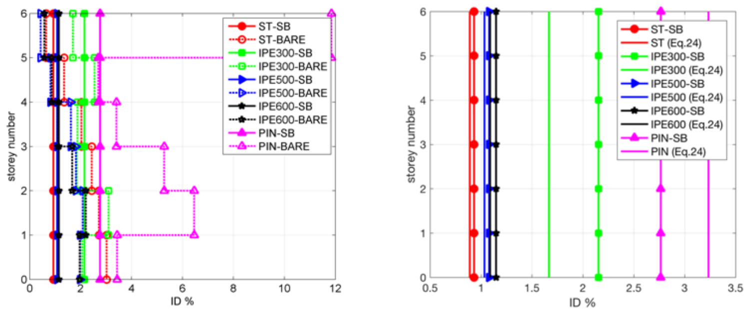
Main results

Figure 6a displays the along the height profile of peak inter storey drift ratio $ID_i = (\delta_{\max,i}/h)\cdot 100$ ($\delta_{max,i}$ is the maximum lateral inter storey drift under earthquake ground motion of the *i*th floor). As expected, the frame structures equipped with strong-back systems exhibit a constant ID_i value along the entire building height. At first approximation, by neglecting the higher mode contributions and considering a roof displacement equal to twice the ordinate of the spectral displacement, the constant ID value may be roughly estimated by using the following equation:

$$ID_{SB} = \frac{2\cdot S_d(T_1)}{N\cdot h}\cdot 100 \tag{24}$$

Figure 6a displays the along the height profile of peak inter storey drift ratio ID_i for all analyzed systems as obtained from the numerical simulations. Figure 6b compares the predictions given by Eq. 24 with the results of the numerical simulations.

Figure 7 displays the envelope of the shear diagrams (both positive and negative peak values) in the bare frame structures and in the frame



(a) (b)

Figure 6: Peak inter-storey drift profiles: (a) Comparison between bare frames (solid line with full markers) and corresponding frames with strong-back systems (dotted line with empty markers). (b) Comparison between frames with strong-back system and predictions of Eq. 24.

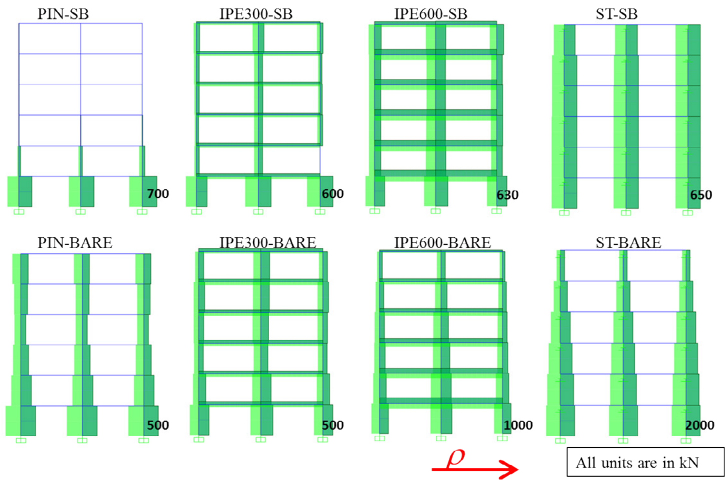


Figure 7: Comparisons of shear diagrams for bare frames (figures in the bottom alignment) and corresponding frames with strong-back systems (figures in the top alignment).

structures with strong-back systems, respectively. Comparison of the shear diagrams leads to the following observations:

- For both cases of the shear type (ST) structure and the structure with stiff beams (IPE600) the presence of the strong-back tends to level out the values of the shear with quite remarkable reductions at the lower stories with respect to the corresponding bare frames.

- For the structures with pinned beam to column connections the presence of the strong-back leads to an along the height distribution of mutual actions quite different with respect to the case of ST structures. In particular, a large concentration of actions in the first storey is observed, whereas the upper stories remain essentially unloaded. A sound interpretation of these observations will require a specific study which is out of the scope of the present work.

- The frame structures with relatively flexible beams (IPE300) tend to behave like the corresponding pinned structures even though the peak values of the shear in the bottom storey are reduced.

Conclusion

The present work presented the first results of a study aimed at assessing the seismic behavior of frame structures with strong-back systems, namely a vertical truss system having the function of imposing a linear lateral deformation to the main frame when subjected to lateral loads. In particular here the attention has been focused on the evaluation of the mutual actions exchanged between the frame and the strong-back. For this purpose a shear type frame idealization has allowed to obtain analytical expressions of along the height profile of the mutual actions. It is found that for the case of uniform lateral stiffness distributions the mutual action at the top storey becomes quite large (approximately the same order of magnitude of the base shears).

A possibility to minimize the mutual actions is to size the columns so that under a given set of lateral forces the frame would develop a nearly linear lateral deformation profile. An analytical expression of such lateral stiffness profile has been obtained assuming a shear type schematization.

Finally some numerical simulations have been carried out considering the influence of actual beam flexibility and real recorded earthquake. It is found that for the case of pinned beam to column connections the actual profile of the mutual actions significantly differs from the one analytically obtained for the case of shear type frames. For that case, a very large mutual action at the first storey is exchanged between the frame and the strong-back. The sound explanation of this behavior will be the objective of a future work.

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