

# Spatial Distribution of Hysteretic Energy in Reinforced Concrete Moment Resisting Frames



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**Abstract** Nowadays, structural engineers perceive that conventional force-based seismic design method is not still the unique way of designing structures subjected to ground motions. The reason is that it does not consider inelastic displacement, plastic structural behavior and duration of seismic motion. At the present time, there are new and popular alternatives like displacement-based method, in which the aforementioned issues are mostly handled. Energy-based approach is another convenient tool to examine the seismic response of structures under seismic action and probably the best way to include duration of ground motion within the analysis. In this approach, the energy input to the structure should be dissipated through inelastic action (hysteretic energy) and damping. Hence it is an important challenge to obtain the distribution of hysteretic energy within the building in order to develop energy-based design and analysis tools. Such studies have been conducted for steel frames previously, but not extensively for RC frame structures. Accordingly, this study is focused on the story-wise and component-wise distribution of hysteretic energy in RC moment resisting frames. For this purpose, RC frames with different number of stories and bays are designed according to the 2018 Turkish Seismic Code. Then the designed frames are modeled by using lumped plasticity approach. The developed models are subjected to a set of strong ground motion records and the distributions of hysteretic energy for each frame and analysis are obtained. The results indicate that it is possible to set up some rules for the hysteretic energy distribution in RC frames that can be used in energy-based design and analysis procedures.

**Keywords** Energy-based design · Hysteretic energy · RC frame · Story-wise distribution · Component-wise distribution

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## 1 Introduction

In the conventional force-based design approach, the basic aim is to provide enough strength capacity in component and system levels by employing design base shear force. The design calculations are based on linear elastic analysis, in which inelastic action is accounted for by using force reduction factors. However, numerous past studies have shown that force-based approach has many pitfalls and it is not a good candidate to make the comparison between inelastic seismic demand and capacity in a realistic manner. In order to overcome this issue, there are two alternatives: displacement-based approach and energy-based approach. Displacement based approach has been very popular for the last two decades and it has been implemented to most of the current seismic codes as an alternative design tool since target demand of structure and performance of seismic action are quantifiable in this approach. On the other hand, energy-based approach is rather old when compared to displacement-based counterpart, as it has been first proposed by Housner in 1956 [1] who simply stated that input energy of an earthquake should be less than energy dissipation capacity of the structure. After this date, numerous studies have been conducted on energy-based design and analysis procedures, but practical and widely accepted energy-based methodologies have not been developed for real-life structures yet. The main reason is the complexity of energy-based parameters since energy is simply the product of force and displacement, two basic parameters that are used in alternative design approaches. Hence it is not easy to propose practical calculation procedures that can be easily conceived by practicing engineers. On the other hand, it should also be noted that the use of energy parameters means total control on force and displacement terms. Hence one can monitor the full inelastic seismic response and compare it with the corresponding seismic capacity throughout the complete duration of ground motion. Moreover, energy-based design and analysis procedures have an indisputable superiority when one has to deal with base-isolated structures or structures with external damping mechanisms.

This study aims to contribute to the development of practical energy-based design and assessment methodologies to be implemented in the future generations of seismic codes and standards. For this purpose, code-compliant reinforced concrete (RC) moment resisting frames (MRFs) are employed in this study to investigate the spatial distribution of hysteretic energy through members and stories when the planar frame models are subjected to a series of ground motion records. The design of the frames is carried out by using the latest version of Turkish Building Seismic Code, which can be abbreviated as TBSC-18 [2]. The reason for selecting RC frames in this study is two folds: First, most of the related past studies have been carried out on steel frame structures. There are few studies that are focused on the energy-based response of RC frame structures [3–8]. Second, in Turkey, RC frame buildings constitute the majority of the building stock, therefore development of energy-based design and assessment procedures for this construction type seems to be more crucial.

## 2 Energy-Based Seismic Response

Energy-based approach is focused on the premise that energy demand during seismic action can be quantified and energy supply of the structure can be provided. In order to obtain the energy terms, equation of motion for an inelastic damped Single-Degree-of-Freedom (SDoF) system should be integrated with respect to the relative displacement

$$\int m\ddot{u}du + \int c\dot{u}du + \int f_s du = - \int m\ddot{u}_g du \quad (1)$$

where  $m$ ,  $c$  and  $f_s$  are the mass, damping coefficient and restoring force of the SDoF oscillator, respectively, and  $\ddot{u}_g$  is the ground motion acceleration. For an inelastic SDoF system, restoring force is a function of relative displacement  $u$ , i.e.  $f_s = f_s(u)$ . Hence it should be represented by a hysteresis model that simulates the cyclic force-displacement response of the inelastic SDoF system.

The above equation is defined as “the relative energy balance equation” since the ground excitation term is on the right-hand side of the formulation. This physically represents a SDoF system with a fixed base subjected to an equivalent lateral earthquake force  $m\ddot{u}_g$ . On the other hand, if this term is taken to the left-hand side of the equation, then Eq. (1) can be written as

$$\int m\ddot{u}_t du + \int c\dot{u}du + \int f_s du = 0 \quad (2)$$

where  $\ddot{u}_t$  denotes the total acceleration. This version is known as “the absolute energy balance equation” and it physically represents a SDoF system with a moving base subjected to the ground excitation  $m\ddot{u}_g$  from the base. This model considers both the relative deformation of the frame and the rigid body translation due to ground displacement  $u_g$ .

The differences between these two different interpretations of energy response has been discussed in literature in a detailed manner. Starting with the early works of Uang and Bertero [9], the results of numerical analysis reveal that there is not a significant difference in seismic energy response when these two approaches are compared within the period range 0.3–5.0 s. This is practically the range of periods in which most of the structural systems reside. Bruneau and Wang [10] stated that the relative energy equation is physically more meaningful since all the internal forces are computed by using the relative displacements and velocities. In addition, Kalkan and Kunnath [11] claimed that the response statistics obtained by using the relative energy approach is more rational when near-fault ground motion records are used in the analyses. Since the RC frame models considered in this study have periods within the range 0.3–5.0 s, both energy approaches can be used since they are deemed to yield similar results in terms of energy response. However, relative energy approach seems to be more practical since it considers relative kinematic terms that can be directly extracted from the output data of the used finite element structural analysis

program. Hence relative energy approach, i.e. Equation (1) is selected in this study. This equation can also be written by using the abbreviations of the relative energy terms as

$$E_K + E_D + E_A = E_I \quad (3)$$

where  $E_K$ ,  $E_D$ ,  $E_A$  and  $E_I$  stand for the relative terms of kinetic energy, damping energy, absorbed energy on the left-hand side and input energy on the right-hand side of the equation. Absorbed energy is composed of two sub-terms: recoverable elastic strain energy ( $E_S$ ) and irrecoverable hysteretic energy ( $E_H$ ). When a structural system behaves beyond the linear elastic range under ground motion excitation, energy input to the structure should be dissipated by both  $E_D$  and  $E_H$  (or in other words, inelastic displacement). At the end of ground motion duration, since  $E_K$  and  $E_S$  die out, the summation of  $E_D$  and  $E_H$  gives  $E_I$ . This shows the importance of  $E_H$  distribution within the structure during seismic action since it is directly related with the deformations and damage levels of the members. In the case of Multi Degree of Freedom (MDoF) systems, the same energy balance equation can be used by replacing single-valued kinematic terms with vectors and other terms ( $m$ ,  $c$  and  $f_s$ ) with matrices.

There are many studies that have been focused on the estimation of  $E_I$  in the literature, so it is simple to estimate the energy demand of a given structure. However, there exist fewer research on the determination of  $E_H$  since it is a more complex energy parameter and it is not an easy task to determine the energy dissipation characteristics of structural members. On the other hand, it is crucial to determine the spatial distribution of  $E_H$  within the structure under different ground motion excitations in order to estimate energy demand and then compare it with energy supply. For this purpose, in this study, multi-story RC MRFs are modeled and analyzed by using nonlinear time-history analysis (NLTHA), as it is explained in the following sections.

### 3 Code-Compliant RC MRF Buildings

In this study, 3, 5, 7 and 9 story code-compliant RC MRF building models with different numbers of bays (for 5-story models only) are selected to examine the  $E_H$  response by using NLTHA. The following sub-sections summarize the details in design and numerical modeling of the frame building models.

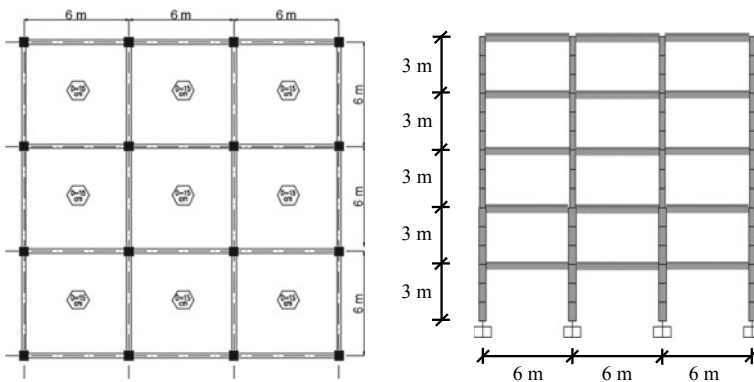
### 3.1 Seismic Design of the RC Frame Buildings

The building models are designed and detailed to satisfy the requirements according to Turkish Standards TS-500 [12] and TBSC-18. The reason for selecting code-compliant buildings is to examine the distribution of inelastic behavior in well-designed RC frame buildings so that energy-based design principles can be proposed by taking the force-based design rules as reference for comparison.

All RC frame buildings are constructed as 3-D numerical models with elastic section properties as dictated by the seismic code. The plan and elevation views of one of the selected RC frame buildings are shown in Fig. 1. In total, six frame models are considered with different number of stories and bays: RCFS3B3 (3 stories and 3 bays), RCFS5B2 (5 stories and 2 bays), RCFS5B3 (5 stories and 3 bays), RCFS5B4 (5 stories and 4 bays), RCFS7B3 (7 stories and 3 bays) and RCFS9B3 (9 stories and 3 bays). Span length in all of the models is considered as 6 m whereas story height is taken as 3 m. Applied dead loads and live loads are taken from the standard TS-498 [13]. In accordance with the design regulations, concrete and steel grades are selected as C25 ( $f_{ck} = 25$  MPa) and S420 ( $f_{yk} = 420$  MPa).

Designed frame buildings are assumed to be located at a site with peak ground acceleration (PGA) equal to 0.4 g and site class ZD (stiff clay and medium compact sand). The design of the frames is carried out by using the conventional force-based design approach in TBSC-18 for a seismic hazard level with a return period of 475 years and Life Safety performance level. Frames are designed as high ductile with structure system behavior factor  $R = 8$  and overstrength factor  $D = 3$  according to the seismic code.

The dynamic properties of the frame buildings are determined by using cracked section rigidities for beams and columns of the frame models as enforced by the TBSC-18. All beams are designed as T-sections with constant dimensions of 25 cm  $\times$  45 cm and 100 cm effective flange width. The amount of beam reinforcement varies



**Fig. 1** Floor plan and elevation views of one of the selected frame buildings models: RCFS5B3 (5 stories and 3 bays)

**Table 1** Beam member design details

Beam section (cm × cm)	Beam Rebar (top)	Beam Rebar (bottom)	Flange reinf	Transverse reinf. (confinement zone)	Transverse reinf. (central zone)
25 × 45	3φ16	3φ16	6φ10	φ10/10	φ10/15
	3φ18	4φ16	6φ10	φ10/10	φ10/15
	4φ18	4φ16	6φ10	φ10/10	φ10/15

**Table 2** Column member design details

Column section (cm × cm)	Column Rebar	Transverse reinf. (confinement zone)	Transverse reinf. (central zone)
35 × 35	8φ20, 8φ25	φ10/10	φ10/20
40 × 40	8φ20, 8φ25	φ10/10	φ10/20
50 × 50	12φ20	φ10/10	φ10/20
60 × 60	16φ20	φ12/10	φ12/20
70 × 70	20φ20	φ12/10	φ12/20

depending on the design load on the member as shown in Table 1. For column design, square sections are used for which dimensions and reinforcement details vary from story to story for all frames. Member section details used in the design of the frames are presented in Table 2. Slab thickness is taken as 15 cm for all frame buildings.

All design checks have been performed for the considered frame buildings, including the capacity design principles (especially strong column-weak concept) and it has been verified that strength and deflection requirements are all satisfied.

### 3.2 Numerical Modeling of the RC Frame Buildings

Numerical modeling and dynamic analyses of the RC frame buildings are carried out by using the SAP2000 finite element analysis software [14]. During dynamic analysis, nonlinear materials properties are assigned by using the lumped plasticity approach. This is achieved by placing nonlinear hinges at two ends of linear elastic frame members. Plastic hinge length ( $L_p$ ) is assumed as half of the section depth in the considered direction as recommended by TBSC-18. The selection of the lumped plasticity approach is rational for MRFs since maximum moment and in turn concentration of plastic deformation usually take place at the end zones of beam and column members.

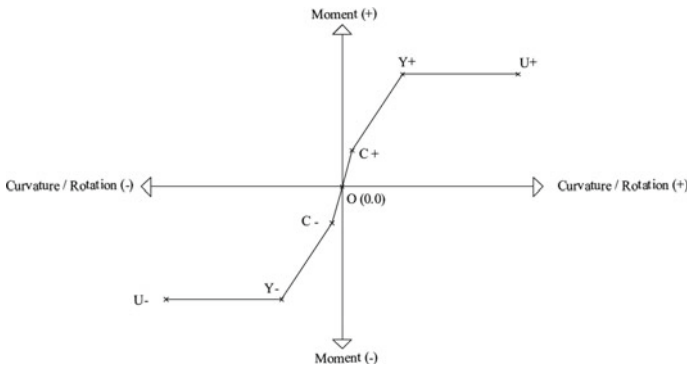
Different hysteresis models can be employed to simulate the cyclic nonlinear behavior at hinge elements. In this study, Takeda hysteresis model [15] is used since it is known to simulate the nonlinear cyclic behavior of RC frame members with

sufficient accuracy and reasonable simplicity. Since the frame members are expected to be in high ductility class due to design requirements, premature failure modes like shear and bond slip are eliminated. In addition, design calculations show that the columns are subjected to low-to-medium ranges of axial load. Hence flexural behavior seems to be dominant under seismic action for all frame members. These points verify the use of Takeda model in this study to simulate the global behavior of building frame models under seismic action.

Takeda hysteresis model uses a trilinear backbone curve for force-displacement (or moment-curvature) relationship with three limit states: tensile cracking (C), yielding of longitudinal reinforcement (Y) and ultimate capacity (U) in both positive and negative directions (Fig. 2). There are 16 rules in this hysteresis model to determine the instantaneous stiffness at a loading or unloading step. The details of these rules can be obtained from [15]. Takeda model has already been implemented in SAP2000 software.

In order to use Takeda model in nonlinear hinges, moment-curvature information of all beam and column end sections are determined and 3 limit states (C, Y and U) are defined in terms of moment-curvature pairs to form the backbone curve of the selected model. The limit state values for all members and frames are provided in Azizi [16].

It should also be mentioned that only the planar interior frames are considered for dynamic analysis instead of 3-D numerical models used in design due to symmetrical plan layout in both orthogonal directions. Rayleigh damping is used in analysis for a damping ratio of 5% in which the first two natural frequencies of the frame models are considered.



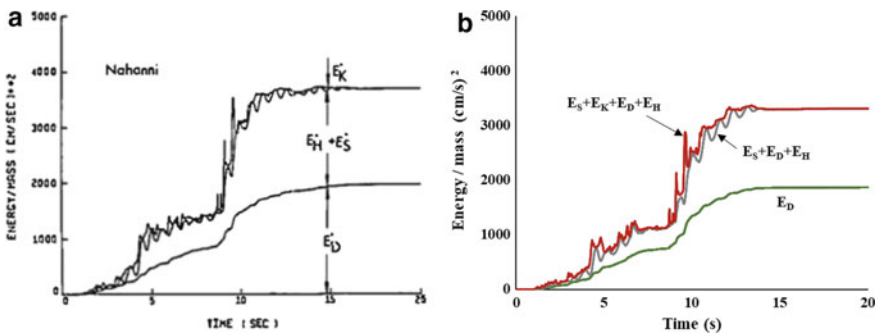
**Fig. 2** Backbone curve and limit states for Takeda hysteresis model

### 4 Dynamic Analyses Applied to the Model Frames

The selected frame models are analyzed by using NLTHA. For this purpose, 20 strong ground motion records are employed in two groups: local records and global records. Local set is composed of 10 records from past major earthquakes that occurred in Turkey (labelled as L1-L10) whereas global set has the same number of records which had been recorded during major earthquakes in different parts of the world (labelled as G1-G10). In the selection of ground motion records, the main criterion is to have ground motion variability in terms of duration, intensity and frequency content. The major characteristics of the ground motion records are provided in Table 3.

Before performing dynamic analysis, all ground motions are scaled for each frame building model according to the ordinate of the 5% damped target design spectrum at the fundamental period of that building in order to impose the same acceleration demand from different records at design earthquake level.

In this study, 6 different frame models are subjected to 20 ground motion records, which means in total 120 NLTHA are conducted and energy parameters given in Eq. 3 are calculated by considering the nonlinear response history of each frame subjected to a specific ground motion record. But before conducting the NLTHA, a benchmark case study is carried out in order to verify the energy calculations within the software. For this purpose, the 10 story RC frame developed by Zhu [3] is modeled in SAP2000 and then the benchmark frame is subjected to Nahanni, Canada (1985) ground motion record. The energy time history results obtained by Zhu in the original research are compared with the ones obtained in this study. As seen in Fig. 3, the energy values and the trends are very close to each other.



**Fig. 3** Time history of energy components for 10 story RC frame under Nahanni (1985) record **a** by Zhu [3], **b** by the verification study



**Table 3** Characteristics of the selected global and local ground motion records

Label	Event	Country	Year	Location	M	PGA (g)	PGV (cm/s)
L1	Horasan	Turkey	1983	Horasan	6.7	0.13	36.9
L2	Erzincan	Turkey	1992	Erzincan	7.3	0.47	92.1
L3	Dinar	Turkey	1995	Dinar	6.1	0.32	40.6
L4	Kocaeli	Turkey	1999	Yarımca (NS)	7.8	0.32	79.6
L5	Kocaeli	Turkey	1999	Yarımca (EW)	7.8	0.23	84.7
L6	Kocaeli	Turkey	1999	Duzce	7.8	0.34	60.6
L7	Duzce	Turkey	1999	Duzce (NS)	7.3	0.41	65.8
L8	Duzce	Turkey	1999	Duzce (EW)	7.3	0.51	86.1
L9	Bingol	Turkey	2003	Bingol	6.4	0.51	34.5
L10	Ceyhan	Turkey	1998	Ceyhan	6.2	0.23	29.8
G1	Imperial Valley	USA	1979	El Centro Array #5	6.5	0.37	95.9
G2	Montenegro	Yugoslavia	1979	Ulcinj	7.0	0.24	47.1
G3	Loma Prieta	USA	1989	Hollister	7.0	0.37	62.8
G4	Manjil	Iran	1990	Abhar	7.3	0.21	55.4
G5	Cape Mendocino	USA	1992	Petrolia	7.0	0.66	89.5
G6	Northridge	USA	1994	Slymar	6.7	0.37	118.9
G7	Northridge	USA	1994	Jensen F.P	6.7	0.42	106.2
G8	Kobe	Japan	1995	JMA	6.9	0.83	90.7
G9	Chi-Chi	Taiwan	1999	TCU074	7.6	0.60	74.6
G10	Tabas	Iran	1978	Tabas	7.3	0.24	47.1

## 5 Energy-Based Results from Dynamic Analyses

This section is devoted to the presentation of energy-based results from nonlinear dynamic analyses conducted for the proposed 6 RC frame models under the selected 20 ground motion records. Although all of the results in terms of energy parameters have been obtained from the NLTHA, only the ones related with the distribution of hysteretic energy within the frames are discussed in this section as the main focus of this study. For detailed discussion regarding the variation of input energy and the ratio of hysteretic to input energy, one can refer to Azizi [16]. In this section, the energy results are presented in terms of the story-wise and member-wise distributions of  $E_H$  demand within the given frame.

### 5.1 Story-Wise Distribution of Hysteretic Energy

Dissipated  $E_H$  at column and beam ends are summed for each story and then divided by the total  $E_H$  of the frame to find the normalized  $E_H$  demand of each story. The distribution of story-wise  $E_H$  to total  $E_H$  ratio (i.e.  $E_{SH}/\Sigma E_H$ ) over the height of the frame under global ground motion set are shown in Fig. 4, in which MG represents mean value of the results. It is worth stating that the results obtained for 5-story frame models with different number of bays are very close to each other. Hence, model RCFS5B3 is used to represent the results of all 5-story frame models. The mean values of the  $E_{SH}/\Sigma E_H$  ratio for global and local records are provided in Table 4.

From evaluation of results obtained for the story-wise distribution of  $E_H$ , the following comments can be concluded:

- General trend of the results shows that distribution of  $E_H$  over the height of the structure depends on both ground motion characteristics and structural properties.
- For model RCFS3B3, ratios for nearly all ground motions seem to be close to each other whereas the ratio values have more scatter for models with higher number

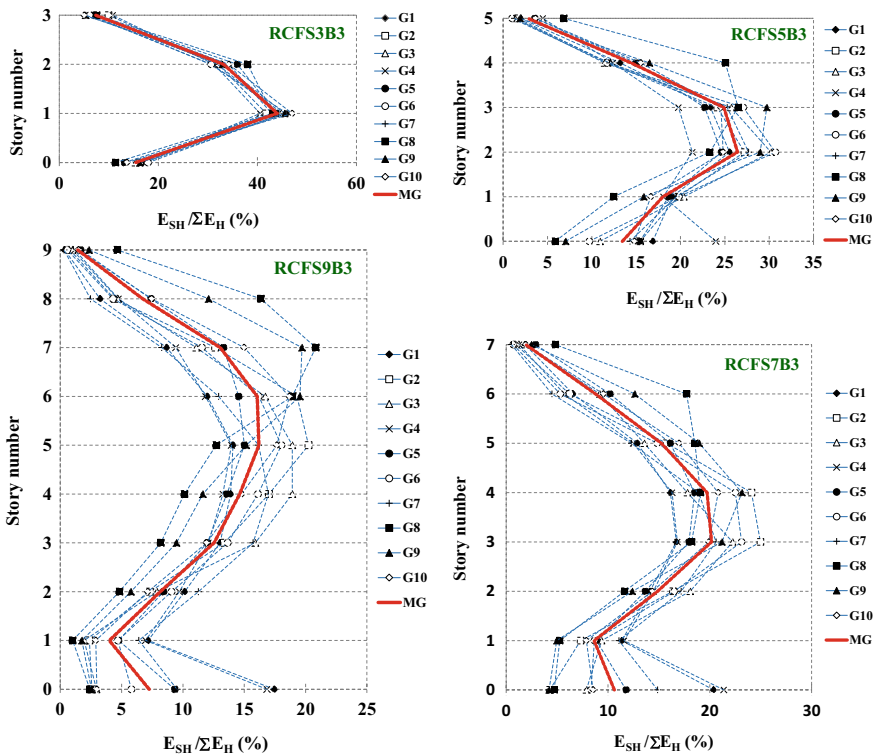


Fig. 4 Story-wise variation of  $E_{SH}/\Sigma E_H$  ratio for 3, 5, 7 and 9-story buildings by using the global set of records

**Table 4** Mean values of  $E_{SH}/\Sigma E_H$  (%) for global and local record sets

Story	RCFS3B3		RCFS5B3		RCFS7B3		RCFS9B3	
	Global	Local	Global	Local	Global	Local	Global	Local
9	–	–	–	–	–	–	1.4	1.1
8	–	–	–	–	–	–	6.7	4.9
7	–	–	–	–	1.9	2.3	13.0	10.2
6	–	–	–	–	8.8	7.8	16.1	13.8
5	–	–	2.8	3.7	15.2	13.9	16.2	15.9
4	–	–	14.5	15.0	19.7	18.7	14.6	15.5
3	7.4	8.7	24.9	23.7	20.2	19.5	12.5	13.7
2	33.2	33.5	26.4	25.1	14.9	15.1	8.2	9.5
1	44.0	43.5	18.0	17.4	8.6	9.5	4.0	5.3
Base	15.4	14.3	13.4	15.1	10.6	13.3	7.2	10.0

of stories. This shows that story-wise distribution of  $E_H$  is less sensitive to ground motion characteristics for low-rise buildings.

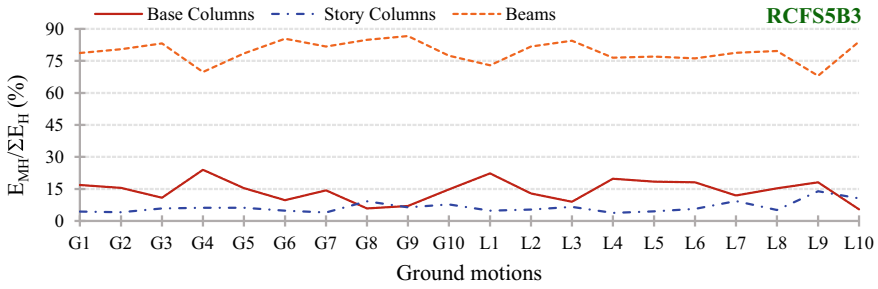
- According to Table 4, 75% of  $E_H$  is dissipated in the 2nd and 3rd stories for RCFS3B3. This value decreases to 45% for RCFS5B3, 25% for RCFS7B3 and 15% for RCFS9B3. In addition,  $E_H$  in the base columns decreases from 15% for RCFS3B3 to 9% for RCFS9B3. It means that as number of stories increases,  $E_H$  propagates from the lower stories to the middle and upper stories.

## 5.2 Member-Wise Distribution of Hysteretic Energy

The dissipated  $E_H$  at each beam and column end is divided by the total  $E_H$  to obtain the ratio of member-wise (beam or column)  $E_H$  to total  $E_H$  ( $E_{MH}/\Sigma E_H$ ) for all ground motions as a measure. The percentages of total  $E_H$  dissipated by the base columns, story columns and story beams for all records are presented in Fig. 5 for model frame RCFS5B5 only, since the results for the remaining 3, 7 and 9-story frame models have similar trends. In addition, the mean values of  $E_H$  percent dissipated by beam and column members for global and local ground motion sets are provided in Table 5.

Evaluating the member-wise distribution of  $E_H$  results given in Fig. 5 and Table 5, the following comments can be concluded:

- Calculated mean values of  $E_H$  dissipated at column and beam ends for all frame models indicate that 78% of total  $E_H$  is dissipated by beam hinges whereas 9% is dissipated by column hinges in the stories and 13% is dissipated by the hinges at the base columns. It is also observed that at a beam-column joint, 10% of the  $E_H$  is dissipated by columns whereas 90% is dissipated by the beams.



**Fig. 5** Member-wise  $E_H$  distribution for frame model RCFS5B3

**Table 5** Mean  $E_{MH}/\Sigma E_H$  values (%) for structural members under global and local records

Frame	Base columns		Story columns		Beams	
	Global	Local	Global	Local	Global	Local
RCFS3B3	15.4	14.3	16.4	18.9	68.2	66.8
RCFS5B2	12.8	14.2	5.7	6.5	81.6	79.3
RCFS5B3	13.4	15.1	5.9	7.0	80.7	77.9
RCFS5B4	13.9	15.5	6.1	7.2	80.1	77.3
RCFS7B3	10.6	13.3	8.3	8.7	81.0	78.0
RCFS9B3	7.2	10.0	7.7	7.0	85.1	83.0

- As stated before, strong column-weak beam criterion (i.e. sum of column moment capacities at a joint is 20% more than the sum of beam moment capacities) has been ensured in the design of frame models in the context of TBSC-18. Accordingly, the total column moment capacity to total beam moment capacity ratios at the joints of frame model RCFS3B3 are 1.7 on the average whereas the same ratios get values around 2.0 for the other frame models. These values seem to be directly reflected on  $E_H$  ratios as seen on Table 5. The mean  $E_{MH}/\Sigma E_H$  value for story columns in frame model RCFS3B3 is approximately 18%. However, for the other frames, this ratio is observed to take values around 7%. Hence it can be clearly stated that the ratio of dissipated  $E_H$  by columns to dissipated  $E_H$  by beams is sensitive to the ultimate moment capacities of column and beam sections at joints.
- Although the column moment capacity to beam moment capacity ratio is more than 1.2 for all frame models according to the force-based design requirements as stated above, the columns did not behave all in the linear elastic range and they exhibited hysteretic energy due to inelastic deformation. It could be concluded that the strong column-weak beam requirement (i.e. ratio of 1.2 in many seismic codes) does not guarantee the elastic behavior of columns but it induces a ductile beam-column failure mechanism.

- Referring to Table 5, it is observed that as the frame model becomes more flexible (i.e. number of stories increases), dissipated  $E_H$  is transferred from the base columns to the beams.

Mean values of  $E_{MH}/\Sigma E_H$  are presented at each nonlinear hinge for frames RCFS3B3, RCFS5B3, RCFS7B3 and RCFS9B3 in Fig. 6. The following comments are based on these  $E_H$  distributions:

- Comparison of the  $E_{MH}/\Sigma E_H$  ratios shows that the percentage of  $E_H$  dissipated by exterior and interior members are not the same.
- The values indicate that the  $E_{MH}/\Sigma E_H$  ratios for exterior beam hinges are greater than the ones for interior beam hinges. The inverse trend occurs for columns, i.e. the  $E_{MH}/\Sigma E_H$  ratios for exterior column hinges are less than the ones for interior column hinges. This means that the  $E_H$  demand of exterior beams and interior columns are more critical than the  $E_H$  demand of the interior beams and exterior columns of the same story in RC MRFs.
- If one finds the difference of  $E_{MH}/\Sigma E_H$  ratios between exterior and interior beam hinges for all stories and takes their average value, it can be observed that the  $E_H$  dissipated by exterior beam hinges is 30% more than the  $E_H$  dissipated by the interior beam hinges for RCFS3B3, 15% for RCFS5B3, 8% for RCFS7B3 and 4% for RCFS9B3. This means that as the number of stories increases, the difference in  $E_{MH}/\Sigma E_H$  ratios between exterior and interior beam hinges decreases and the distribution becomes quite regular.
- Since, the amount of  $E_H$  dissipated by columns is small and majority of the  $E_H$  is dissipated by beams, it is possible to ignore the difference in  $E_{MH}/\Sigma E_H$  ratios between exterior and interior column hinges.
- Overall, member-wise distribution of  $E_H$  is directly influenced by the moment capacities of beam and column sections at a joint whereas the dependence is slight for different ground motion sets. This shows that it is possible to propose practical energy-based design rules to control the distribution of inelastic action within a frame structure.

## 6 Conclusions

This study is a preliminary attempt to promote energy-based design and assessment approaches to next generation seismic codes. In order to achieve this task, seismic response in terms of energy should be examined thoroughly. This has somewhat been accomplished for the input energy demand since there are many past studies in the literature focusing on the estimation of the parameter  $E_I$ . However, research about the distribution of  $E_H$  demand within a structure has not been studied to the same extent, especially for RC frame buildings. Hence this study plans to give contribution to the findings regarding the story-wise and member-wise distributions of  $E_H$  in RC frame buildings. This study contains some assumptions and simplifications as stated

in the previous sections. The following conclusions can be stated without ruling out these limitations:

0.2	0.1	0.0	0.0	0.1	0.3
0.8	2.6	RCFS3B3		2.6	0.6
0.1	0.3	3.8	3.8	0.3	0.1
5.1	3.9	3.8	3.9	5.1	
0.7	2.7	2.7	2.7	0.7	
0.1	0.6	6.2	6.2	0.6	0.1
8.2	6.4	6.2	6.4	8.2	
0.1	0.3	0.3	0.3	0.7	
3.1	4.6	4.6	4.6	3.1	

0.3	0.2	0.1	0.2	0.2	0.4
0.1	0.5	RCFS5B3		0.6	0.1
0.0	0.1	2.0	2.1	0.1	0.0
2.2	2.1	2.0	2.1	1.9	2.5
0.2	0.5	0.5	0.5	0.2	
0.1	0.2	3.8	4.0	0.2	0.1
4.2	4.0	3.8	4.0	3.7	4.4
0.0	0.2	0.2	0.2	0.2	0.0
0.0	0.1	4.2	4.3	0.1	0.0
4.4	4.3	4.2	4.3	4.1	4.6
0.0	0.1	0.1	0.1	0.1	0.0
0.2	0.5	2.7	2.7	0.5	0.2
2.9	2.7	2.7	2.7	2.6	3.0
0.0	0.0	0.0	0.0	0.0	0.0
3.2	3.5	3.5	3.5	3.2	

0.1	0.0	0.0	0.0	0.1	0.1
0.1	0.4	RCFS9B3		0.4	0.1
0.0	0.0	0.9	0.8	0.0	0.0
1.0	0.8	0.9	0.8	0.9	0.9
0.2	0.5	0.5	0.5	0.2	
0.1	0.1	1.9	1.8	0.1	0.1
2.2	1.8	1.9	1.8	1.9	2.1
0.1	0.4	0.4	0.4	0.1	
0.0	0.1	2.6	2.6	0.1	0.0
2.7	2.5	2.6	2.6	2.6	2.7
0.0	0.1	0.1	0.1	0.0	
0.0	0.1	2.5	2.6	0.1	0.0
2.7	2.6	2.5	2.6	2.5	2.7
0.1	0.1	0.1	0.1	0.1	0.1
0.1	0.2	2.3	2.3	0.2	0.1
2.4	2.3	2.3	2.3	2.3	2.4
0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.1	2.0	2.0	0.1	0.1
2.0	2.0	2.0	2.0	2.0	2.1
0.0	0.0	0.0	0.0	0.0	0.0
0.2	0.2	1.2	1.2	0.2	0.2
1.2	1.2	1.2	1.2	1.2	1.2
0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.2	0.5	0.5	0.2	0.1
0.5	0.5	0.5	0.5	0.5	0.5
0.1	0.0	0.0	0.0	0.0	0.1
1.8	1.9	1.9	1.9	1.8	

0.2	0.1	0.0	0.0	0.1	0.1
0.1	0.6	RCFS7B3		0.6	0.1
0.0	0.1	1.1	1.1	0.1	0.0
1.3	1.1	1.1	1.1	1.1	1.3
0.2	0.6	0.6	0.6	0.2	
0.1	0.2	2.0	2.1	0.2	0.1
2.5	2.2	2.0	2.1	2.1	2.6
0.1	0.5	0.5	0.5	0.5	0.1
0.0	0.0	3.0	3.2	0.0	0.0
3.3	3.2	3.0	3.2	3.1	3.4
0.0	0.1	0.1	0.1	0.1	0.0
0.0	0.1	3.1	3.3	0.1	0.0
3.3	3.3	3.1	3.3	3.1	3.5
0.0	0.1	0.1	0.1	0.1	0.0
0.2	0.3	2.2	2.3	0.3	0.2
2.3	2.3	2.2	2.3	2.2	2.4
0.0	0.0	0.0	0.0	0.0	0.0
0.2	0.4	1.2	1.2	0.4	0.2
1.2	1.2	1.2	1.2	1.2	1.3
0.1	0.0	0.0	0.0	0.0	0.1
2.6	2.7	2.7	2.7	2.6	

Fig. 6 Mean values of  $E_{MH}/\Sigma E_H$  at the joints of all frame models with three bays under global ground motion set

- Considering the general trend of the story-wise distribution of  $E_H$  results, it is observed that distribution of  $E_H$  over the height of structure depends on both ground motion characteristics and structural properties. The dependency of the  $E_{SH}/\Sigma E_H$  ratio to ground motion characteristics become more obvious as the number of stories increases (i.e. structure becomes more flexible).
- The  $E_{SH}/\Sigma E_H$  ratio of ground story and lower stories decrease as the number of stories increases. Hence  $E_H$  demand shifts from lower stories to upper ones as structure becomes more flexible (i.e. going from 3-story to 9-story frame models). This reveals that the second mode of the structures should also be considered in an energy-based design or assessment methodology (during estimation of the story-wise distribution of hysteretic energy) for mid-rise and high-rise structures.
- The results of this study show that in a well-designed RC moment-resisting frame, approximately, 70–85% of the  $E_H$  is dissipated by beams, 8–18% is dissipated by story columns and 7–15% is dissipated by base columns. Consequently, it seems that in a ductile RC moment-resisting frame, majority of  $E_H$  is dissipated by beams, which is a verification of the intended behavior in force-based capacity design of RC frame structures. It also seems that the percent of  $E_H$  dissipated by columns or beams strongly depends on the ultimate moment capacities of columns and beams sections at the joints of the frame.
- Although strong column-weak beam criterion is considered in design of frame models, the columns exhibited inelastic behavior. This observation indicates that assigned safety factor of 1.2 for the ratio of column moment capacity to beam moment capacity does not guarantee elastic behavior for columns and it causes a ductile beam-column failure mechanism. This is not surprising since the frame models are designed for Life Safety performance level, for which controlled damage is allowed. The important point is that the percentage of inelastic action is very limited in columns when compared to beams. This is a verification in the force-based seismic design process for ductile behavior.
- Member-wise distribution of  $E_H$  in the same story shows that  $E_H$  is distributed uniformly between interior members. This may also be verified for exterior members. However, comparing interior and exterior members together indicates that  $E_{MH}/\Sigma E_H$  values at the ends of members are not equal. In addition to this, the  $E_H$  demand of exterior beam hinges are generally more than interior beam hinges whereas the  $E_H$  demand of interior column hinges are more than exterior column hinges. This difference in  $E_H$  demand between interior and exterior members becomes more pronounced in low rise RC frame building. Hence it can be stated that as number of stories increases,  $E_H$  is distributed more uniformly in the same story.
- The dynamic analysis results obtained in this study reveal that the spatial distribution of  $E_H$  within a frame is highly affected by the number of stories whereas it does not seem to be influenced by the number of bays.
- The aforementioned results regarding story-wise and member-wise distributions of  $E_H$  can assist to estimate the role of each member to dissipate a certain amount of energy in an energy-based design methodology and the capacities of members can be arranged in accordance with this demand. So, it can be finally stated that

energy-based parameters are promising in order to estimate the distribution of energy demand in a RC frame structure. This leads to the motivation that simple yet robust energy-based approaches can be developed and implemented to the future releases of seismic codes if the energy dissipation capacities of the members can be determined in a satisfactory manner.

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