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Effects of deviation in materials' strengths on the lateral strength and damage of RC frames

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Abstract. The real behavior of the RC structures constructed based on the assumed specifications of the used materials is matched with the designed ones when the assumed and the applied specifications in construction are the same. Despite in the construction phase of the reinforced concrete (RC) structures always it is tried to implement the same specifications of materials as given in the executive drawings, but considering the unpredicted/uncontrolled parameters that affect the specification of materials, always there is a deviation between the constructed and the designed materials' specifications. The objective of this paper is to submit a guideline for the evaluation of the strength and damage to the existing RC structures encountered deviation in materials' strengths. To achieve this goal, the lateral strength (plastic behaviors) and damage to twenty-five RC moment-resisting frames (MRFs) are studied by applying the inelastic analysis. In this study, a couple of concrete and reinforcement strengths' deviations are investigated. The obtained results indicate that in general, there is a semi-linear relationship between the deviation in the strength of reinforcement and the changes in the lateral strength values of the MRFs. The relative effect of the deviation in the strength of reinforcements is more than the relative effect of the deviation in the concrete strength on the damage rate. The obtained results could be a guideline for the engineers in the survey of the existing buildings encountered deviation in materials' strengths during their construction phase.

Keywords: deviation; strength; material; inelastic; vulnerability; damage

1. Introduction

Allocation of the materials' strengths and the mechanical properties of materials is the first step in the designing process of structures. In the designing process, the strength level of structures should be in the elastic behavior range or overtake the elastic phase by considering a response modification factor (which considers the seismic behavior of the structure), based on the purpose of the design. One of the main preconditions to satisfy the design's expectations and achieve the targeted purposes is the coincidence of the designing assumptions and the constructed mechanical properties of the used materials. This control may be carried out by non-destructive tests (Moshtagh and Massumi 2011).

In the last decade, some nonlinear numerical simulation methods to model the behavior and the damage to the structures counter to the earthquake lateral loading are proposed. Massumi *et al.* (2013) offered a damage index for the seismic damage assessment of RC buildings based on the variation of the nonlinear fundamental period obtained by field tests. They assessed precisely and expeditiously the seismic situation of a couple of existing RC buildings that have experienced an earthquake by employing their proposed damage index. Heo and Kunnath (2013) proposed a damage-based approach for the performance-based seismic assessment of RC frame-structures. A new methodology for structural damage assessment was developed by them that utilizes response information at the material level in each section fiber. The material level damage parameter is combined at the member, story and structural level using weighting factors. The damage model was used to compare the performance of two typical 12-story frames that have been designed for different seismic requirements. A probabilistic approach was finally used to quantify the expected seismic performance of the building. Habibi and Asadi (2017) have proposed a drift-based index to estimate the damage to RC MRFs with the setback. The inelastic dynamic time-history analysis was performed on several frames with different types of setbacks subjected to various earthquake records and the damage to them computed by the Park-Ang DI. They showed that the proposed damage index is capable to estimate the damage of setback frames. Merter (2017) presented an energy-based method to determine the earthquake safety of RC frame structures. This method is based on the comparison of plastic energy capacities of the structures with plastic energy demands obtained for selected earthquake records to perform the nonlinear time history analyses. Earthquake

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plastic energy demands are determined from nonlinear time history analyses and hysteretic behavior of earthquakes is converted to monotonic behavior by using nonlinear moment-rotation relations of plastic hinges and plastic axial deformations in columns. Earthquake safety of the RC frame is assessed by using plastic energy capacity graphs and earthquake plastic energy demands. Gharehbaghi (2018) proposed a procedure for the seismic design of RC frame structures to minimize the construction cost by considering the uniform damage distribution over the height of the structure due to earthquake excitations. This procedure is structured in the framework of an optimization problem, and the initial construction cost is chosen as the objective function. He presented a damage pattern based on the concept of global collapse mechanism. Nouban and Sadeghi (2018) proposed a macro element-based algorithm to analyze the RC 1D structural members (SMs) under monotonic or cyclic combined loading. The 1D SMs are discretized into macro-elements (MEs) located between the critical sections and the inflection points. The critical sections are discretized into fixed rectangular finite elements (FRFE). The proposed algorithm has been validated by the results of experimental tests carried out on full-scale RC structural members. Zhao *et al.* (2018) proposed a modified rigid body spring model (RBSM) and used it to analyze the damage and failure process of reinforced concrete (RC) structures. In this model, the concrete is represented by an assembly of rigid blocks connected with a uniform distribution of normal and tangential springs to simulate the macroscopic mechanical behavior of concrete, besides, the steel bars are equally dispersed into rigid blocks as a kind of homogeneous axial material, and an additional uniform distribution of axial and dowel springs is defined to consider the axial stiffness and dowel action of steel bars.

It is possible to identify and quantify the extent of damage of RC members under monotonic, cyclic or fatigue loading through a non-dimensional factor known as "damage index" (DI). The DI can be defined either for the entire structure as a global DI or as a local DI at the member level in its critical section.

Existing damage indices are based on different characteristics such as the number of cycles (Shah 1984, Chung 1989, Oh 1991), stiffness (Lybas and Sozen 1977, Roufaiel and Meyer 1987), ductility (Park *et al.* 1987, Gupta 2001, Bertero and Mahin 1976), energy (Banon *et al.* 1981, Park and Ang 1985, Darwin and Nmai 1986, Meyer and Kratzig 1988, Sadeghi 1998, Sadeghi and Nouban 2016), local damage (Sadeghi and Nouban 2017) and global damage based on the assessment of local materials' damages (Amziane and Dubé 2008).

The well-known DI proposed by Park and Ang (1985) is based on the plastic-hinge approach and consists of both deformation and energy terms. The DI proposed by Park and Ang was criticized by Abbasnia *et al.* (2011) for not considering the shear contribution in the DI.

Although the construction of RC structures is based on the approved for construction (AFC) drawings and the designing assumptions, any change in the mechanical properties such as the materials' strength deviation is probably due to the in-situ mixing and undesirable condition of concrete transfer. Furthermore, inaccurate reinforcing, casting, curing and maintenance of concrete lead to an inappropriate seismic behavior, especially in the post-elastic phase of loading.

In this paper, the lateral strength level and vulnerability of RC-MRFs are studied considering the deviations of concrete and reinforcements' strengths. A number of multi-span, multi-story RC-MRFs are designed and their inelastic behaviors are studied by considering a set of given strength deviations in the analyses. Finally, the results are discussed in detail and are compared via employing tables and graphs. The obtained results can be employed in the re-evaluation of the existing structures. In the evaluation process of the existing RC structures, the obtained results could be helpful in decision making because they reflect the actual nonlinear behavior and defects of the RC structures. Considering these results would help engineers to select rehabilitation or reconstruction of the existing RC structures encountered deviation in materials' strengths.

2. The research significance

Since the adequate range of safety considered by different codes is affected adversely, the working stress design (elastic theory), ultimate strength design and limit state design will not be practical while the materials' strengths are out of expectations. In other words, the strength, serviceability and ductility provisions are not enough to satisfy the expectations.

Furthermore, to evaluate the possible upper and lower bands of likely strength of structural members especially in RC structures, some assumptions have been considered in the derivation of the strength equations. Hence, the levels of possible strength of members used in various types of design calculations are defined. It is obvious that the ideal strength, dependable strength, strength and overstrength levels could be affected adversely by the deviation in materials' strengths. Note that the concrete properties such as the modulus of elasticity, distribution of cracks and the concrete behavior against the repeated loading could be affected intensely by the materials' strengths deviation.

In general, the deviation in reinforcing bars' strength could change the confinement of concrete and the balance of forces in all sections of an RC member. Hence, all of these effects could lead to an unpredictable static and seismic behavior of RC structures and decrease the amount of the dissipated energy due to the seismic loads intensely.

3. Characteristics of the studied structural models

Twenty-five highly ductile RC-MRFs with a different number of stories and spans, placed in a high seismic risk zone, designed in accordance with the Iranian code of practice for the design of the seismic-resistant building "BHRC" (2005) have been studied. The selected structures are 1-, 2-, 4-, 6- and 10-story RC-MRFs having 1, 2, 3, 4 and 5 spans as illustrated in Fig. 1.

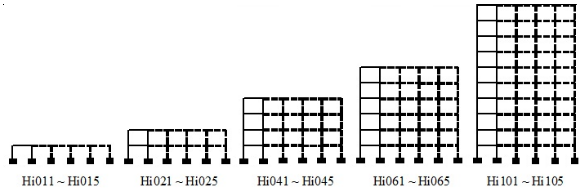

Fig. 1 Selected MRFs (the added stories and spans are shown by dashed lines)

The MRFs with the number of stories and spans that are utilized in common and ordinary buildings are selected to include a wide range of redundancy (3 to 150 redundancy). Hence, this range of redundancy eradicates all misgivings about the effects of redundancy on the results. The height of stories and the length of spans of the modeled structures are 3m and 4m, respectively. Moreover, the intermediate level of importance and the soil class II have been considered to design the modeled MRFs in accordance with the BHRC code (2005) and the part 6 (structural loadings) of the Iranian national building codes (2005). The two-way concrete slabs are utilized for the ceilings with common details. Finally, the structural modeling, analysis and the design of the MRFs have been performed applying the ACI-318-02 code (2002), SAP2000 (2011) and IDARC2D-5 (1990) software.

4. Labeling system utilized to identify the MRFs

A labeling system is utilized to identify easily the MRFs, consisting of two characters and three digits, illustrated as "Hixyz". Where, "H" represents the seismic zone and the ductility degree of the structure, which in this research it is assumed to be high seismic risk zone and designed to be highly ductile (R=10). The second character "i" stands for the inelastic state of the structure, and represents the type of the employed analysis. The middle part two-digit "xy" indicates the number of stories (01 to 10) and the last digit "z" indicates the number of spans (1 to 5).

5. Applied pushover analysis

To evaluate the inelastic behavior of the MRFs, the pushover analysis with the inverted triangular lateral load distribution has been carried out by IDARC2D-5 software, which is a program for the inelastic damage analysis of buildings in a high precision (Valles *et al.* 1999). This precision is due to the modeling assumptions. By employing this software the models reflect the actual seismic behavior of structures.

5.1 Nonlinear incremental static method applied in the analyses

A usual method used to calculate the strength of structures subjected to the incremental lateral loading is the incremental static analysis (ISA) method. Incremental dynamic analysis (IDA) is too much time consuming and energy consuming. Comparison between the ISA and the IDA methods made by Massumi *et al.* (2004) illustrated that the ISA method can be used to calculate the strength and overstrength of the RC-MRFs with a reasonable accuracy. In the present study, the ISA method is applied to analyze the structures subjected to the fixed vertical loading and the incremental lateral loads with the patterns of inverse triangular loads in the structure's height as well as for all levels of strength of concrete and reinforcements. The overall response curves of the structures under the incremental lateral loading together with the bilinear idealized responses of them by employing Park's recommendation (based on the elastic-plastic system with the equivalent reduced stiffness) were obtained. The maximum values of the equivalent overall lateral displacements are limited to 3% of the height of the structure. Note that, among the different criteria used for the collapse of the structures (which are explained in details by Massumi 2004), only the main criterion of the maximum displacement was applied and controlled in this research. Therefore, all of the overall behavior responses of the structures are ended at a displacement equal to 3% of the height of the structures. The selection of this value for the maximum lateral displacements of the structures is based on the research performed in this field by Massumi (2004), Fischinger and Fajfar (1990), as well as ATC (1995). As highlighted by Massumi (2004), the effect of different values of the maximum lateral displacement on the strength and overstrength is negligible.

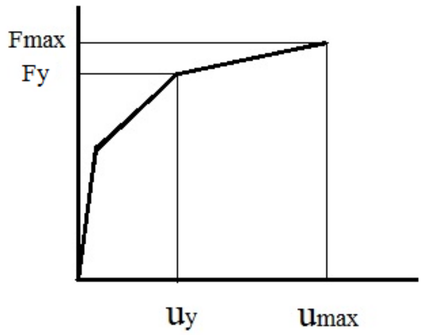

Fig. 2 Response model used in IDARC2D-5

5.2 The model utilized to simulate the ideal seismic behavior

In this study, a three-parameter Park model has been utilized to model ideal seismic behavior by incorporating stiffness degradation, strength deterioration, non-symmetric response, slip-lock, and a trilinear monotonic envelope (Park *et al.* 1987). This model traces the response of an RC member precisely as it changes from one linear stage to another, depending on the imposed deformations (see Fig. 2).

6. Determination of the damage index (DI)

Using damage indices and/or damage functions is a common way to assess the seismic vulnerability of structures. In this research, the well-known DI proposed by Park-Ang (Eq. (1)) has been modified and utilized to evaluate the effect of deviations in the materials' strengths on the vulnerability of RC-MRFs. This DI is applied because it is based on the dissipated energy in the structural members and their deformations as described below. It also weights them according to their importance for the global assessment of the structures.

$$DI_{P\&A} = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u P_y} \int dE_h \tag{1}$$

where:

δ_m : Maximum experienced deformation,

δ_u : The ultimate deformation of the element,

P_y : The yield strength of the element,

$\int dE_h$: The energy absorbed by the element during the response history,

β : The model constant parameter that is proposed 0.1 for nominal strength deterioration (Park and Ang 1975).

Since the inelastic behavior of RC members are confined to plastic zones in IDARC2D-5 software (Kunnath *et al.* 1992), the following modification (illustrated in Eq. (2)) to the original model was introduced.

$$DI = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_h \tag{2}$$

where:

θ_m : The maximum rotation attained during the loading history,

θ_u : The ultimate rotation capacity of the section,

θ_r : The recoverable rotation when unloading,

M_y : The yield moment,

E_h: The dissipated energy in the section.

Story DI is calculated by the following equation

$$\mathrm{DI}_{\mathrm{Story,j}} = \sum_{i=1}^{i=m} (\lambda_{\mathrm{ji}_{\mathrm{member}}} . \mathrm{DI}_{\mathrm{ji}_{\mathrm{member}}}) \tag{3}$$

with

$$\lambda_{\mathrm{ji}_{\mathrm{member}}} = \left(\frac{\mathrm{E_{ji}}}{\sum_{i=1}^{i=m} \mathrm{E_{ji}}}\right)_{\mathrm{member}} \tag{4}$$

and the overall DI

$$\mathrm{DI}_{\mathrm{Overall}} = \sum_{j=1}^{j=n} \lambda_{\mathrm{Story,j}} . \lambda_{\mathrm{Story,j}}) \tag{5}$$

with

$$\lambda_{\mathrm{Story,j}} = \left(\frac{\mathrm{E_j}}{\sum_{j=1}^{j=n} \mathrm{E_j}}\right)_{\mathrm{Story}} \tag{6}$$

Table 1 Interpretation of overall DI (Park and Ang 1975)

Damage degree	Physical appearance	Overall DI	Building status
Collapse	Partial or total collapse of the building	>1.0	Loss of building
Severe	Extensive crashing of concrete, the disclosure of buckled reinforcement	0.4-1.0	Beyond Repair
Moderate	Extensive large cracks, spalling of concrete in weaker members	<0.4	Repairable
Minor	Minor cracks, the partial crushing of concrete in columns	-	-
Slight	Sporadic occurrence of cracking	-	-

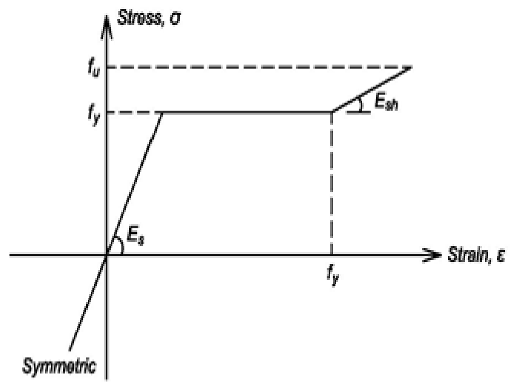

Fig. 3 The employed stress-strain curve of reinforcements (Valles *et al.* 1999)

and with

$$\mathrm{E_j} = \sum_{i=1}^{i=m} \mathrm{E_{ji}} \tag{7}$$

where:

$\lambda_{\mathrm{ji}_{\mathrm{member}}}$: The energy weighting factor of the member "i" of story "j",

$\mathrm{DI}_{\mathrm{ji}_{\mathrm{member}}}$: DI of the member "i" of story "j",

$\mathrm{E_{ji}}$: The absorbed energy by the member "i" of story "j",

$\mathrm{E_j}$: The total absorbed energy by the story "j",

DI: The Park-Ang damage index, DI classification of calibration is illustrated in Table 1 (Park and Ang 1975).

7. Materials' strength deviations

All frames in this study have been designed applying the characteristic compressive strength of concrete (f_c') of 24 MPa and yield strength (f_y) of 300 MPa for reinforcing bars. To assess the effects of deviation in materials' strengths on the lateral strength of the members and vulnerability of structures, the MRFs have been analyzed with considering a range of ±25% deviation in concrete strength (f_c' = 24±6 MPa) a range of ±20% deviation in reinforcements' strength (f_y = 300±60 MPa). The strength deviation of concrete has been applied with increments of 2 MPa (i.e. f_c' = 18, 20, 22, 24, 26, 28 and 30 MPa). The strength deviation of reinforcements has been applied with increments of 20 MPa (i.e., f_y = 240, 260, 280, 300, 320, 340 and 360 MPa). Note that the concrete properties such as the modulus of elasticity, tensile strength and strain

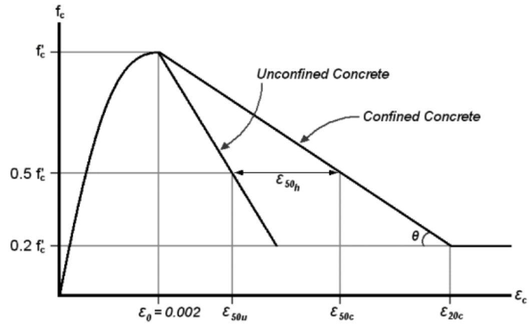

Fig. 4 The employed stress-strain curve of confined and unconfined concretes (Park and Paulay 1975)

corresponding to the ultimate compressive strength are affected by the concrete strength deviation, whereas, the reinforcements' properties up to yielding points of reinforcements are not affected by the reinforcements' strength deviation. Thus, changes in concrete properties considered according to the equations of ACI-318-02 (2002) and ASTM and the constitutive laws of materials proposed by Valles *et al.* (1999) and Park and Paulay (1975), as illustrated in Figs. 3 and 4, are employed in this research.

8. Inelastic incremental static analysis

The lateral strength of the analyzed MRFs has been evaluated by the inelastic incremental static analysis "ISA" (pushover). In this procedure, Park-Ang DI has been utilized to assess the vulnerability of the analyzed MRFs. Besides, in this study, the constant gravitational loads and inverted triangular lateral load distribution are applied to analyze the MRFs applying the aforementioned various materials' strengths. The response of MRFs and the corresponding damage indices are extracted in each step of the analysis. Finally, the response curve of each MRF applying the incremental lateral load is extracted and idealized to a bilinear response curve according to the Park (1989) proposal (based on an elastoplastic system having equivalent reduced stiffness), while the maximum lateral displacements "drifts" (δ)max of MRFs are limited to 3% of their heights (h), i.e., (δ/h)max = 3%.

Note that for all of the studied MRFs, amongst the common failure criteria, only the maximum lateral displacement criterion has controlled the pushover analyses. For this reason, the response curves of structures are limited to 3% of their height. This rate of lateral displacement is selected based on the studies of Massumi (2004), Fischinger and Fajfar (1990), and ATC (1995). According to these studies, the effect of maximum lateral displacement on the lateral strength will be negligible if the maximum lateral displacement is larger than the lateral displacement corresponding to the yield point (i.e., about 1% of the height of the structure).

The overall response curves of Hi105 MRF (i.e., an MRF having 10 stories and 5 spans) under the incremental lateral load and its idealized bilinear response are shown in Fig. 5.

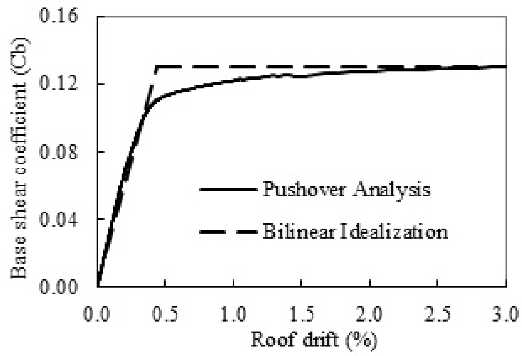

Fig. 5 The overall response (ISA) of Hi105 MRF (f_y = **300** MPa, f_c' = **24** MPa, Δ_{max} = 0.03H)

9. Analysis of the obtained results

The nonlinear inelastic ISA has been carried out for all of the analyzed structures and the obtained response curves have been idealized as bilinear curves. Then, the values of the parameters representing the strength of MRFs such as the base shears corresponding to the design code of practice, the first yield and the yield point of MRFs (as calculated employing Eqs. (8) to (10)) are extracted from the bilinear response curves. An example of the overall response curve under the incremental lateral load and its idealized bilinear response are shown in Fig. 5.

Finally, the modified Park-Ang DI values are calculated.

$$C_d = (V/W)_{Design} \tag{8}$$

$$C_S = (V/W)_{First\ Yield} \tag{9}$$

$$C_y = (V/W)_{Yield} \tag{10}$$

where:

C_d: The design base shear coefficient,

C_s: The first yield base shear coefficient,

C_y: The yield base shear coefficient, respectively,

V : The base shear,

W: The effective weight of the structure considered for the seismic design.

Note that the different base shear coefficients (C_d, C_s and C_y) are calculated based on the Iranian code of practice for the design of the seismic-resistant building (2005) and the obtained results are shown in Tables 2 and 3.

The results obtained from the analyses of the MRFs with 1 to 5 span show that except for the 1-story MRFs, the number of spans has negligible effects on the strength parameters. Therefore, the DI values reported in Tables 2 and 3 have represented the average values related to the MRFs with different spans.

The average values of strength parameters and DI determined for different MRFs having different stories and spans applying f_y = 300 MPa and f_c' = 24±6 MPa are submitted in Table 2. Similar average values for the same MRFs applying f_y = 300±60 MPa and f_c' = 24 MPa are given in Table 3.

The results given in Table 2 illustrates that the reduction or increment rate of the damage rate of the structures is

Table 2 The average values of strength parameters and DI (f_y = 300 MPa, f_c' = 24±6 MPa)

f_c' (MPa)	Structure's label	C_d	C_s	C_y	DI
18	Hi011~Hi015	0.088	0.387	0.494	0.059
	Hi021~Hi025	0.088	0.215	0.314	0.129
	Hi041~Hi045	0.088	0.161	0.206	0.162
	Hi061~Hi065	0.077	0.128	0.171	0.171
	Hi101~Hi105	0.059	0.096	0.129	0.179
20	Hi011~Hi015	0.088	0.382	0.498	0.040
	Hi021~Hi025	0.088	0.232	0.316	0.122
	Hi041~Hi045	0.088	0.162	0.207	0.168
	Hi061~Hi065	0.077	0.130	0.172	0.188
	Hi101~Hi105	0.059	0.096	0.130	0.194
22	Hi011~Hi015	0.088	0.384	0.502	0.063
	Hi021~Hi025	0.088	0.229	0.319	0.123
	Hi041~Hi045	0.088	0.162	0.209	0.173
	Hi061~Hi065	0.077	0.130	0.173	0.195
	Hi101~Hi105	0.059	0.097	0.131	0.200
24	Hi011~Hi015	0.088	0.382	0.510	0.064
	Hi021~Hi025	0.088	0.230	0.321	0.125
	Hi041~Hi045	0.088	0.165	0.209	0.182
	Hi061~Hi065	0.077	0.130	0.174	0.205
	Hi101~Hi105	0.059	0.098	0.132	0.208
26	Hi011~Hi015	0.088	0.385	0.513	0.068
	Hi021~Hi025	0.088	0.239	0.324	0.126
	Hi041~Hi045	0.088	0.165	0.210	0.186
	Hi061~Hi065	0.077	0.131	0.174	0.209
	Hi101~Hi105	0.059	0.098	0.132	0.218
28	Hi011~Hi015	0.088	0.389	0.515	0.070
	Hi021~Hi025	0.088	0.242	0.326	0.128
	Hi041~Hi045	0.088	0.168	0.211	0.185
	Hi061~Hi065	0.077	0.131	0.174	0.213
	Hi101~Hi105	0.059	0.099	0.133	0.226
30	Hi011~Hi015	0.088	0.390	0.522	0.063
	Hi021~Hi025	0.088	0.243	0.328	0.131
	Hi041~Hi045	0.088	0.168	0.212	0.183
	Hi061~Hi065	0.077	0.130	0.175	0.213
	Hi101~Hi105	0.059	0.099	0.133	0.223

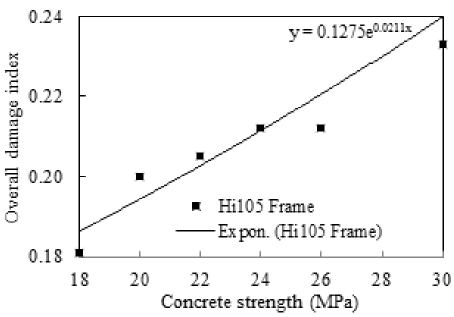

Fig. 6 Overall damage rate versus concrete strength (f_y = **300** MPa)

Table 3 The average values of strength parameters and DI (f_y = 300±60 MPa, f_c' = 24 MPa)

f_y (MPa)	Structure's label	C_d	C_s	C_y	DI
240	Hi011~Hi015	0.088	0.319	0.433	0.074
	Hi021~Hi025	0.088	0.175	0.256	0.131
	Hi041~Hi045	0.088	0.133	0.174	0.205
	Hi061~Hi065	0.077	0.110	0.145	0.227
	Hi101~Hi105	0.059	0.082	0.109	0.254
260	Hi011~Hi015	0.088	0.342	0.459	0.067
	Hi021~Hi025	0.088	0.195	0.275	0.125
	Hi041~Hi045	0.088	0.143	0.186	0.198
	Hi061~Hi065	0.077	0.177	0.154	0.211
	Hi101~Hi105	0.059	0.088	0.117	0.239
280	Hi011~Hi015	0.088	0.327	0.482	0.070
	Hi021~Hi025	0.088	0.22	0.299	0.126
	Hi041~Hi045	0.088	0.156	0.198	0.186
	Hi061~Hi065	0.077	0.123	0.164	0.210
	Hi101~Hi105	0.059	0.093	0.124	0.225
300	Hi011~Hi015	0.088	0.328	0.510	0.064
	Hi021~Hi025	0.088	0.230	0.321	0.125
	Hi041~Hi045	0.088	0.165	0.209	0.182
	Hi061~Hi065	0.077	0.130	0.174	0.205
	Hi101~Hi105	0.059	0.098	0.132	0.208
320	Hi011~Hi015	0.088	0.418	0.531	0.063
	Hi021~Hi025	0.088	0.255	0.337	0.128
	Hi041~Hi045	0.088	0.174	0.221	0.173
	Hi061~Hi065	0.077	0.137	0.183	0.191
	Hi101~Hi105	0.059	0.103	0.139	0.200
340	Hi011~Hi015	0.088	0.451	0.580	0.056
	Hi021~Hi025	0.088	0.275	0.354	0.119
	Hi041~Hi045	0.088	0.181	0.232	0.166
	Hi061~Hi065	0.077	0.142	0.193	0.183
	Hi101~Hi105	0.059	0.108	0.146	0.195
360	Hi011~Hi015	0.088	0.468	0.607	0.054
	Hi021~Hi025	0.088	0.296	0.370	0.117
	Hi041~Hi045	0.088	0.190	0.244	0.157
	Hi061~Hi065	0.077	0.149	0.202	0.176
	Hi101~Hi105	0.059	0.113	0.153	0.181

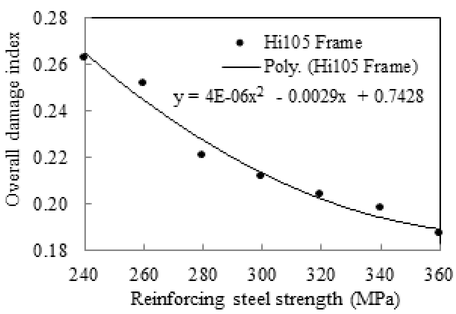

Fig. 7 Overall damage rate versus the strength of reinforcement (f_c' = 24 MPa)

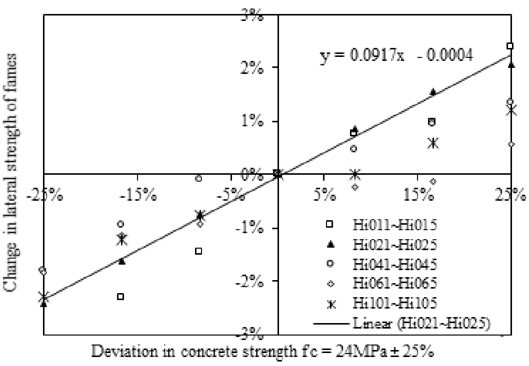

Fig. 8 The change in lateral strength of MRFs versus the concrete strength's deviation (f_c' = 18, 20, 22, **24**, 26, 28, 30 MPa and f_y = **300** MPa)

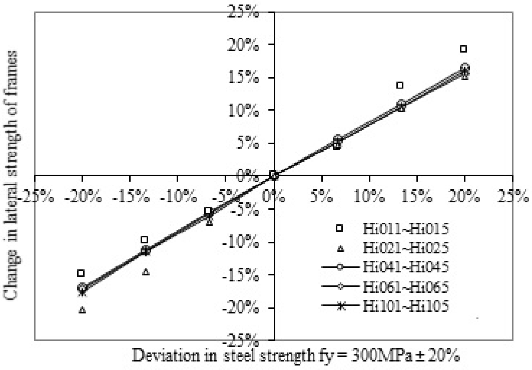

Fig. 9 The change in the lateral strength of MRFs versus the reinforcement strength's deviation (f_c' = 24 MPa and f_y = 240, 260, 280, 300, 320, 340, 360 MPa)

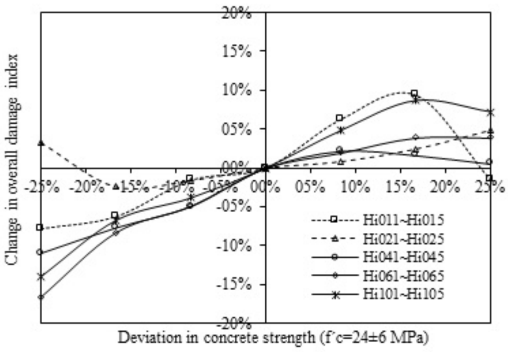

Fig. 10 The change in the overall DI versus the concrete strength deviation (f_c' = 18, 20, 22, 24, 26, 28, 30 MPa and f_y = 300 MPa)

about 40 percent of the reduction or increment of the strength of concrete. Furthermore, the reduction or increment rate of the damage rate of the structures is about 70 percent of the reduction or increment of the strength of reinforcement in a reverse order as reflected in Table 3.

The effects of deviations in materials' strengths on damage rate (overall damage index) for Hi105 MRF applying the maximum drift criterion are shown in Figs. 6 and 7.

As it can be seen from these figures, the patterns of the damage rate variation versus the strengths of concrete and reinforcement are reverse.

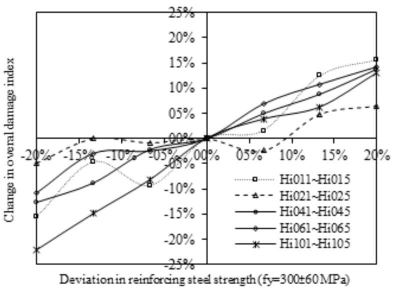

Fig. 11 The change in the overall DI versus the reinforcement's strength deviation (f_c' = **24** MPa and f_y = 240, 260, 280, **300**, 320, 340, 360 MPa)

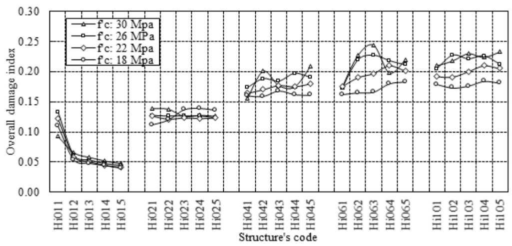

Fig. 12 The effect of concrete's strength deviation on the DI of the analyzed MRFs for a drift of 3%, f_y = 300 MPa

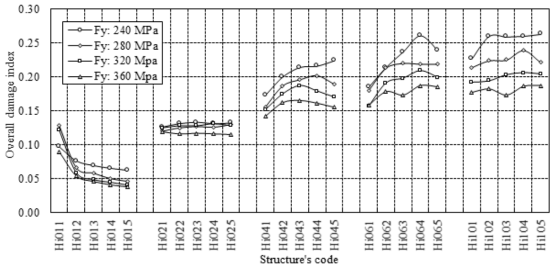

Fig. 13 The effect of reinforcement's strength deviation on the DI of the analyzed MRFs for a drift of 3%, f_c' = 24 MPa

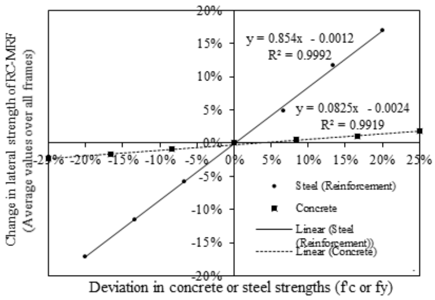

Fig. 14 The average change in the lateral strength versus the materials' strengths deviation

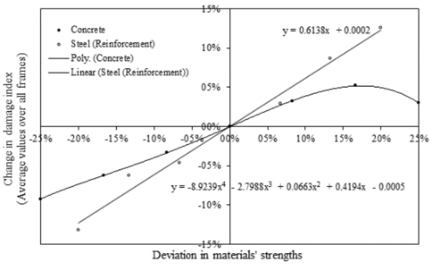

Fig. 15 The average change in the DI versus the materials' strengths deviation

The changes in the lateral strength of MRFs versus the deviation in concrete and the reinforcement strengths have been shown in Figs. 8 and 9 respectively.

As Fig. 9 illustrates, there is a semi-linear relationship between the percentage of deviation in the strength of reinforcement and the percentage of changes in the lateral strength of the MRFs. Besides, the changes in the overall DI versus deviation in the strengths of concrete and of reinforcement have been shown in Figs. 10 and 11, respectively.

Fig. 12 depicts the effect of concrete's strength deviation on the DI of the analyzed MRFs for a drift of 3%, f_y = 300 MPa and f_c' = 18, 22, 26, 30 MPa. As it can be seen from this figure, the overall DI for the structures increases with the increases in f_c' of concrete and the number of stories.

Fig. 13 demonstrates the effect of reinforcement's strength deviation on the DI of the analyzed MRFs for a drift of 3%, f_c' = 24 MPa and f_y = 240, 280, 320, 360 Mpa. As this figure demonstrates, in general, the overall DI increases with the decrease in f_y of reinforcements and it increases with increase in the number of stories.

The comparison of these figures shows that the effects of deviation in the strength of reinforcement are more striking and steady than the effects of deviation in concrete strength is.

To facilitate the evaluation and comparison of all analyses results, the deviation of all MRFs are averaged and are shown in Fig. 14.

According to the obtained results and as it is illustrated in Fig. 14 the change in the lateral strength (the average percentages for all of the simulated R C-MRFs) versus the concrete strength deviation is very close to a linear relationship (with R^2 = 0.9919, where R^2 represents the coefficient of determination) and (by neglecting a very small value of constant value of -0.0024) can be expressed as follows for the studied cases:

$$\Delta V_{\max} = 0.0825\Delta f_c' \tag{11}$$

where:

$\Delta V_{\max}$: Lateral strength change (the average percentages for all simulated RC-MRFs),

$\Delta f_c'$: Percentage of concrete strength deviation.

The change in the lateral strength (the average percentages for the simulated RC-MRFs) versus the reinforcement strength deviation is also very close to a linear relationship (with R^2 = 0.9992) and (by neglecting a very small value of the constant value of -0.0012) can be expressed as follows for the studied cases:

$$\Delta V_{\max} = 0.854\Delta f_y \tag{12}$$

where:

Δf_y: Percentage of reinforcement strength deviation

Based on the obtained results as Fig. 15 illustrates the change in the damage rate (the average percentages for all of the simulated RC-MRFs) versus the concrete strength deviation is very close to a linear relationship for the values of f_c' less than the reference f_c' (24 MPa) but for the values greater than the reference f_c', it is nonlinear and in overall it (with R^2 = 0.9999,) and (by neglecting a very small value of the constant value of -0.0005) can be expressed as follows for the studied cases:

$$\Delta \mathrm{DI} = -8.9239\Delta \mathrm{f}_c'^4 - 2.7988\Delta \mathrm{f}_c'^3 + 0.0663\Delta \mathrm{f}_c'^2 + 0.4194\Delta \mathrm{f}_c' \tag{13}$$

where:

ΔDI: Lateral strength deviation (the average percentages for all simulated RC-MRFs).

The change in the damage rate (the average percentages for the simulated RC-MRFs) versus the reinforcement strength deviation is very close to a linear relationship and (with R^2 = 0.9867) and (by neglecting a very small value of the constant value of -0.0002) can be expressed as follows for the studied cases:

$$\Delta \mathrm{DI} = 0.6138\Delta f_y \tag{14}$$

According to obtained results, the strength deviation of the reinforcement is more influential than concrete strength deviation on the lateral strength and damage rate of structures due to lateral load. It is encouraging because the strength of reinforcement deviation occurs rarely.

10. Conclusions

This study shows that the ratio of the deviation in the strength of reinforcement to the change in the lateral strength is about 90 percent, and the ratio of the deviation in concrete strength to the change in the lateral strength about is 10.

There is a semi-linear relationship between the deviation in the strength of reinforcement and the change in the lateral strength of the MRFs.

In addition, the ratio of the deviation in the strength of reinforcement causes to the change in the damage rate reversely is about 70 percent and the ratio of the deviation in concrete strength to the change in the damage rate is about 40 percent. Although these imposed deviations are not pleasant, the results are encouraging since concrete strength deviation which is more prevalent has not found striking. Furthermore, the results can be used practically to evaluate the existing structures that are subjected to the material strengths' deviations.

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