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An Economical and Behavioral Comparison of Steel Special, Intermediate, and Ordinary Moment Resisting Frames

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Abstract

The Iranian Standard of Seismic Design of Structures No. 2800, IBC 2018, ASCE 7-16 and AISC 341-16 suggest that a response factor, R (also referred to as the R-factor), should be used as a measure to account for some characteristics of the structure such as ductility, degrees of indeterminacy and inherent over-strength capacity. This factor can be applied to buildings of special, ordinary and intermediate Moment Resisting Frame (MRF) systems. This paper presents an investigation of the economic significance of the R-factor on buildings. Previous studies, which focus on construction details of MRF systems, their design criteria and their associated ductility under cyclic loads for domestic manufacturing purposes, are rather limited. The need for such important data became the basis of performance investigation of different moment resisting connections in the present study. A number of 3D regular frames were designed and their regular construction expenses were estimated and compared. Finally, an average of used steel per unit area was derived to provide a measure for quick design inspection of an arbitrary MRF system.

***Keywords**:* Construction Expenses, Building Codes of Practice, Moment Resisting Frames (MRFs), Economical Comparison, Ductility.

1. Introduction

One major aim of structural design procedures, beside safety, is to economize the structure as much as possible, which is the primary basis of most structural design guidelines. It has been trendy to consider design procedures of different design codes to investigate the mechanical and economical behaviors and outcomes of connections and structural members of moment resisting steel frames (e.g. [1-6]). To finalize the design, structural members shall be checked to carry specific loads, so the R-factor is defined which depends on the expected seismic behavior of the structure (e.g. [7-10]). The existing deviation of different R-factors among various structures is due to the difference in their energy dissipation rates in the non-linear structure and material behavior zone. In other words, a structure with higher energy damping capacity in its non-elastic behavior zone has a larger R-factor. Thus, based on the energy dissipation capacity and ductility of the structure, The Iranian Standard of Seismic Design of Structures No. 2800 (called the 2800 Standard hereafter) [7], the International Building Code (IBC 2018) [8], ASCE 7-16 [9] and the American Institute of Steel Construction (AISC 341-16) [10] classify steel moment resisting frames into special, intermediate and ordinary categories, respectively with large, intermediate and small ductility. Each of these classes exhibits its specific different behavior and ductility and requires especial design considerations while it may incorporate a different failure risk. For example, ASCE 7-16 defines the R-factor for special, intermediate and ordinary resisting frames as 8, 4.5, and 3.5, and the 2800 Standard suggests them to be equal to 7.5, 4.5 and 3.5, respectively.
It should be noted that the reason of the main difference in designing such systems rises from different beam-column connections and their panel zone details. Negligence of connection details in the design procedure can lead to undesirable behavior of the structure; consequently, such flaws may result in failure of the structure to absorb sufficient energy

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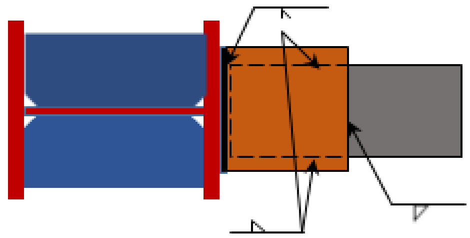
followed by potentially catastrophic outcomes. Moreover, if the R-factor of a structure is incorrectly calculated, the seismic design may be underestimated. Several measures should be considered before choosing the proper R-factor, one of which is to consider the magnitude of the connections' plastic rotation under cyclic loads before rupture of the member. So far, focus has been mainly put on steel moment bearing connections, their cyclic behavior and design criteria proposed for cyclic loading requirements (e.g. [11-15]). Among these investigated connections, a specific type of moment-resisting connection, namely the Welded Flange Plate (WFP) connection, is chosen for use in SMRFs in this study. It is known that the behavior of structural frames, especially moment frames, depends strictly on their connections in terms of their type, design, details and the connecting devices (e.g. [16-18]). For the present study, those design criteria suggested in FEMA 350, which directly consider the welded type of the intended connection, and those in AISC 358-18 that discuss the bolted type, are implemented. Compliance of a designed structure with the recommended specifications is of critical importance. Additionally, since it may be of interest to investors, structural designers and researchers to decrease the costs of the construction e.g. by reducing the consumed material (e.g. [19-36]), one major question is whether or not use of higher-ductility designs for a structure is more economic. It is of interest to obtain the average steel per unit area consumed in moment resisting frames that are designed in compliance to the design provisions in AISC341-16. Another objective of the present study is to examine whether complying with the recommended procedures in the available standards results in reduction of the consumed steel in a specific system. Also, the economic aspect of using structures with higher ductility is examined. Finally, the average consumed steel per unit area for certain three-dimensionally designed Ordinary Moment Resisting Frames (OMRF), Intermediate Moment Resisting Frames (IMRF) and Special Moment Resisting Frames (SMRF) based on the aforementioned codes is reported and compared.

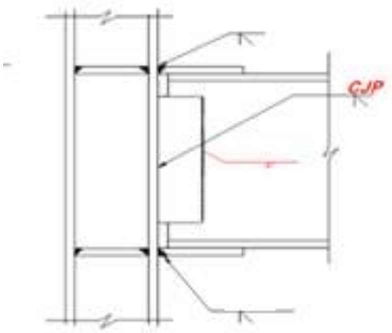
2. Design of Moment Connections to Satisfy Demanded Rotation

AISC 358-18 and FEMA 350 mandate achieving certain rotations under cyclic loadings as an essential requisite for satisfaction of the specified ductility of moment resisting connections. The minimum magnitudes of the recommended rotations are 0.04, 0.02, and 0.01 radian, respectively for special, intermediate and ordinary MRFs. FEMA 289 [14] describes testing requirements and according procedures to perform the test. Based on this standard, "typical loading histories shall consist of a cyclically applied vertical displacement imparted to the end of the beam through actuator(s). This induced a rotational demand on the assembly, similar to that which would be experienced by the beams and columns in a frame subjected to lateral sideway. Initial displacements are within the elastic range of response for the assembly. Displacements are usually applied at very slow strain rates with the specimens held for some period of time at the peak displacement portion of each cycle to allow observation of any damage, however, in some tests, loading is applied at rates that approximate those which would be experienced in a real structure responding to earthquake ground shaking. Several full cycles are repeated at each beam tip displacement level, then the loading is increased and several more cycles are performed. This process is repeated either until failure of the specimen occurs or the limiting displacement of the actuators is reached. The individual test summaries indicate the specific loading protocol employed for each test." Using the data obtained from the test, FEMA 289 formulates correlations to calculate the total rotation θ_{total}, beam rotation θ_b and an empirical yielding displacement δ_y. Required equations and criteria for designing various connections are presented in FEMA 350 which introduces some prequalified connections for special and ordinary MRFs and their corresponding design procedures. Also, construction details are included therein.

The present study aims to select, discuss and assess three connections with desirable cyclic performances for SMRF, IMRF and OMRF structures. The selected connections are designed after FEMA 350. A schematic view of the WFP connection, which is used in special moment frames, is depicted in Figure 1. The manufacturing expenses of the connection are also estimated. To accomplish this objective, three WFP connections were investigated as case studies for the SMRFs.



(a) Top view



(b) Side view

Fig. 1. Schematic View of the used WFP Conection

In the aforementioned connection type no direct connection exists between the beam and column flanges and it is only the cover plates that connect the corresponding counterparts. The cover plates are complete-penetration-groove-welded to the column’s flange and the beam flanges are fillet-welded to the top and bottom cover plates. Figure

1 provides a schematic view of this type of connection. Separated by a small gap, the beam web is connected to the column’s flange using a shear tab which is fillet-welded on the sides of the plate. Doubler plates, which should be parallel with the column web and continuity plates of the upper and lower plates, can be employed if needed. The thicknesses of the upper and lower flange plates are calculated using section plastic moment at the plastic hinge location. This moment is multiplied by a correction factor and used for calculation of the length of the upper and lower plates. Moreover, the welding type, electrode type and weld grinding method are also to be noted. Due to the importance of special moment frames performance under seismic loads, the design procedure must be adequate to assure that the required rotations can be provided, and the required energy dissipation can consequently occur in and by the connections. The obtained rotation can cast doubt on the adequacy of the recommended R-factors if the target rotations are not completely achieved. The Tenth Topic of the Iranian National Building Regulations [37] mandates application of 125% of the design load for design of rigid connections in steel structures. Comparing this requirement with AISC-LRFD requirements that are established upon extensive experimental investigations, it is revealed that the 25% additional load is even less than the value dictated by AISC-LRFD for prequalified connections. The second connection employed in IMRFs structures is the Welded Unreinforced Flange – Bolted Web (WUF-B) connection, which is vastly discussed in AISC 358-18 [38].

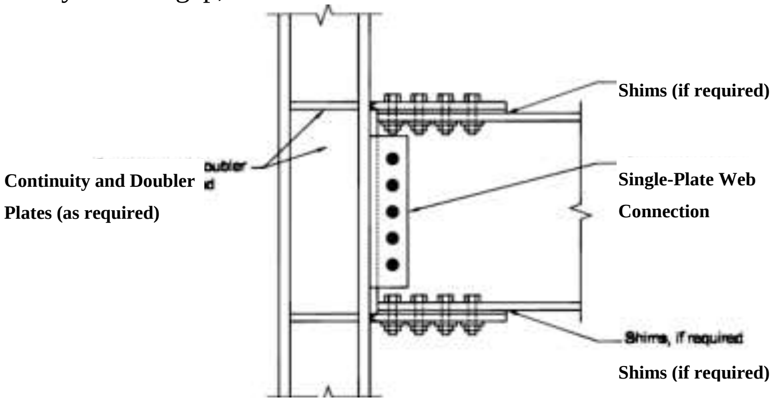


Fig 2. Schematic view of the WUF-B used in the IMRF Structures

The last connection, namely Welded Unreinforced Flange-Welded Web (WUF-W), is one of the prequalified connections introduced in FEMA 350 for OMRF structures. Figure 3 illustrates the typical details of this type of connection.

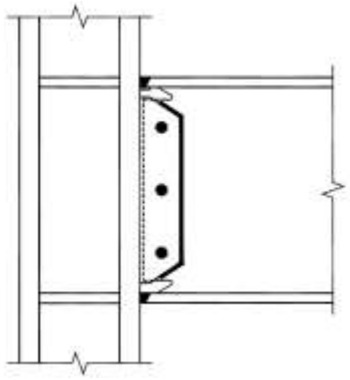


Fig.3. WUF-W connection used in the OMRF Structures

3. Special, Intermediate and Ordinary Steel MRFs

3.1. Geometry

To be able to derive reliable and generalizable results from the study, it is customary to check the design outcomes for several sets of structures with different geometries (e.g. [39-43]). To this end, a total of 45 structures in three groups of special, intermediate and ordinary MRFs were analyzed and designed in compliance with ASCE 7-16 and AISC 358-18 design regulations. Then, the structures were compared based on their average weights of consumed steel per unit area. The typical geometries of the regular 3D structures are given in Table 1.

Table 1
Geometrical Configurations of the Designed Structures

Number of Stories	15						9						5					
Number of Bays in Each Direction	6			3			6			3			6			3		
Span Length in Each Direction (m)	6	5	4	6	5	4	6	5	4	6	5	4	6	5	4	6	5	4

Floor dead and live loads equal 7kN/m^2 and 2 kN/m^2, respectively. The external walls are considered to weigh 7.1 kN/m. Loading was applied according to ASCE 7-16 and the designs were performed in compliance to AISC 341-16 [10] and AISC 360-16 [44]; thus, the basic design procedure was in fact in accordance with the Load and Resistance Factor Design method (LFRD). Accordingly, application of ordinary MRFs is not permitted in high-seismicity hazard regions. Nevertheless, this study is conducted for high-seismicity hazard conditions since one of the objectives is to compare the results obtained from different designs. To make up for this choice of frames, it shall be noted that the other two options, which are permitted for such regions, are also considered. In order to control the inter-story drifts, the International Building Code 2018 [8] was used. The professional structural design software ETABS (Ver. 9) [45] was made use of to analyze and design the structures. AISC-LFRD 2005 [46] is one of the many codes available in the libraries of this software package. Designs of the structural members used in the special, intermediate and ordinary MRFs were performed in complete compliance to AISC-LFRD 2005 [46]. Choosing AISC 360-05/IBC 2006 [47 and 48] as the applied design codes in ETABS results in meeting all design criteria of AISC-LRFD 2005 [46] for the loadbearing elements. After the structures are designed, control of the drifts with code-provided limitations determines whether or not the results are acceptable. Next, the connections are designed for the three types of MRF systems. To fulfill this task, connection details tested through experiments which met the minimum essential performance requirements under cyclic loads were used. The connection types considered in the present study are limited to welded and/or common practice connections in Iran. Design provisions of the connections whose proper performance were verified are stated in FEMA 350 [49] and AISC 358-18 [38]. As a secondary outcome of the designs, the weights of the required profiles, connections and stiffeners, the weight and type of consumed electrodes for welding, the number of necessary tests to confirm satisfactory behavior of the connections and construction costs are taken into account. Finally, the total costs attributed to the three types of the MRFs are compared.

During the design procedure, in some cases it was the drift that governed the final design, so restrictive drift limitations may result in heavier profiles, especially in special MRFs. The second phase of the study is, therefore, intended to investigate the governing criteria (drift vs. strength) in the three categories of structures. Finally, the best MRF is chosen based on the amount of material consumed. It is eventually discussed whether the application of AISC-LRFD codes would lead to any reduction in the construction cost, in terms of required steel material, of SMRFs compared to the other MRFs, when the required ductility is provided.

3.2. Parametric Study

As stated, the forty five structures, whose geometries were given in Table 1, were designed using ETABS. Figures 5 show the weights of the required steel material after the design, respectively for the beams and columns, connections and overall elements and members (i.e. beams, columns and connections altogether) in the structures versus their numbers of stories. All graphs are drawn based on the median amount of steel weight per unit area. Figure 4.c shows that the total steel weight per unit area varies from 84 kg/m^2 to 124 kg/m^2, respectively for ordinary 5- and 15-story structures. For the intermediate 5- and 15-story structures, the weights are between 84 kg/m^2 and 117 kg/m^2, while it varies between 81 kg/m^2 and 112 kg/m^2 for the special 5-story to 15-story structures. It is evident from the figures that the SMRF is lighter than the IMRF and the IMRF is lighter than the OMRF. This may be resulted from the differences in their R-factors which directly affect the input seismic loads. Since drift is the governing criterion in the design of special and intermediate structures, the difference between the required steel material weights for the structures of these two groups is negligible. However, this difference would be more significant if some braces or shear walls were applied (the brace topology and arrangement are meant to be constantly similar in all of the 3 types of structures). In this case, the SMRFs would basically be much lighter than the IMRFs. It can be shown that design of the connections for plastic moment capacity leads to lighter connections in the IMRFs.

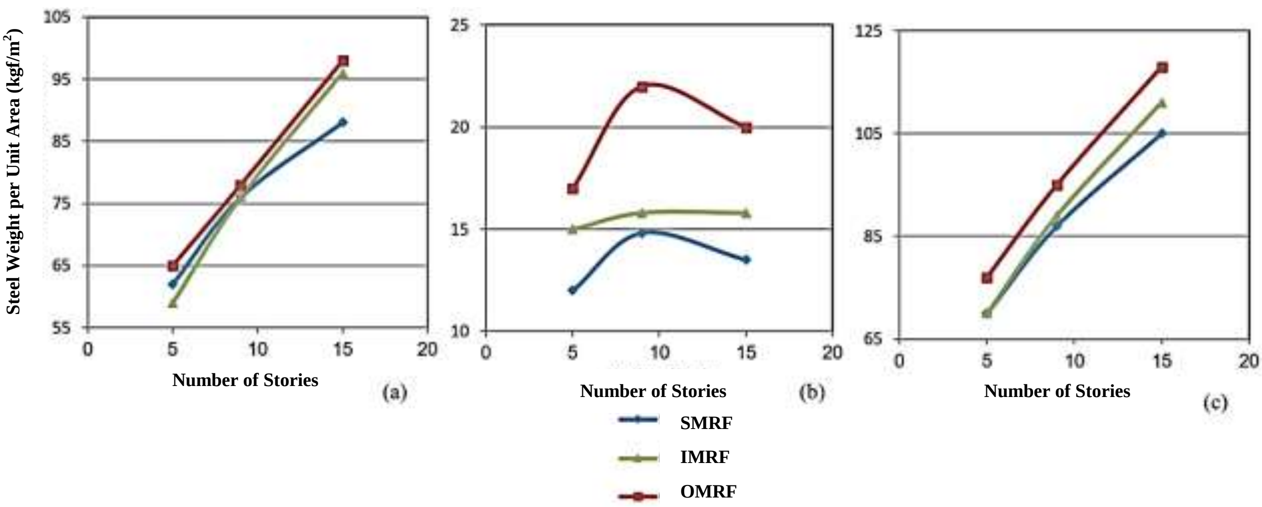


Fig.4. Median steel weight per unit area versus number of stories for SMRFs, IMRFs and OMRFs for (a) Beams and columns, (b) Connections (c) The total amount (i.e. beams, columns and connections)

It can be observed from the graphs that when the number of bays and span lengths are constant, increasing the number of stories leads to increased required material, which results from increased drifts that shall be controlled by increasing the profiles' dimensions in absence of a bracing system. The IMRFs and SMRFs used in the low-rise structures do not show any tangible difference while in the high-rise structures the IMRFs need more steel material. Similar graphs are presented for structures with the same number of stories and same span lengths. Figures 6 shows that increasing the number of spans or, in other words, higher number of structural elements (i.e. beams, columns and moment connections in this study) , results in higher stiffness and indeterminacy of the structures and, therefore, drifts can be more easily controlled. Thus, while the number of columns as well as spans are increased by a suitable arrangement of the columns, optimal use of columns and proper control of drifts and strengths occur simultaneously. Hence, the required steel material weight per unit area for IMRFs and OMRSs will be considerably more than that for SMRFs and, accordingly, SMRFs can be assumed more economical choices than IMRFs and OMRFs in that regard.

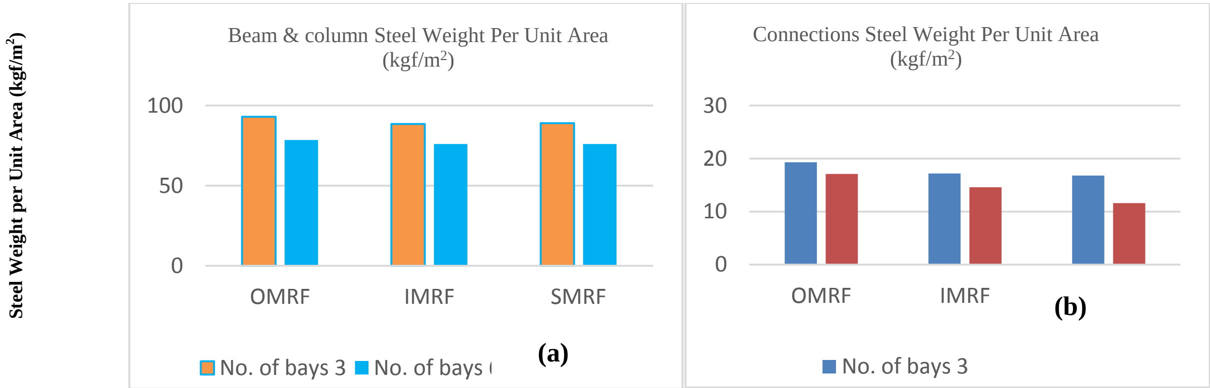


Fig.5. Required steel material weight versus number of bays (a) Beams and columns, (b) Connections

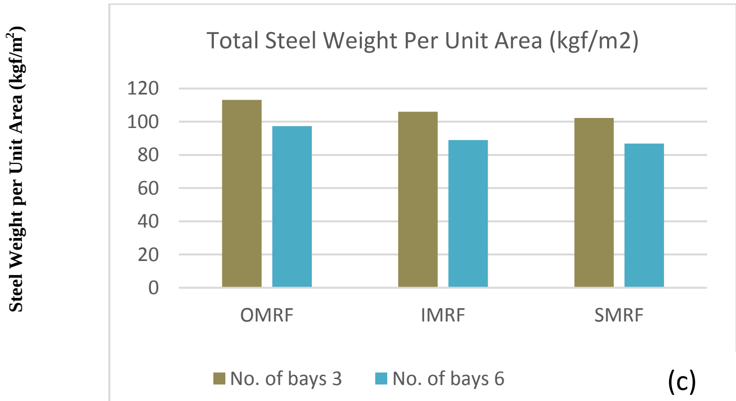


Fig.5. Required steel material weight versus number of bays, (c) The total amount (i.e. beams, columns and connections)

Based on Figure 5, it can be perceived that in case of increase in the number of bays (in the present study from 3 to 6), with constant number of stories and span lengths, a decrease in the required steel weight per unit area is resulted due to the increase of the number of columns and moment bearing connections, which in turn results in higher degrees of redundancy and more stiffness of the frames. Therefore, control of the inter-story drifts (which in SMRFs and IMRFs is the governing criterion) in stiffer frames can be concluded to be easier and, therefore, no additional stiffness shall be provided through heavier beams and columns. In other word, in absence of braces or shear walls, increase of the number of beams and columns leads to enhancement of lateral stiffness of structures, which might be a way to satisfy the drift criterion. Thus, increasing the number of bays can result it decrease in consumed steel per unit area. Moreover, similar graphs are presented for structures with the same number of stories and spans and different span lengths. Figures 6 shows the designed steel weight per unit area for the beams and columns, connections, and the total material required in the structures versus span length. According to Figure 6.c, selection of optimum span length (5 m in the present study) and number of columns leads to optimum designed sections that are able to satisfy the inter-story drift criterion ASCE 7-16 [9] and carry gravity and lateral loads, which brings about additional decrease in required steel weight per unit area..

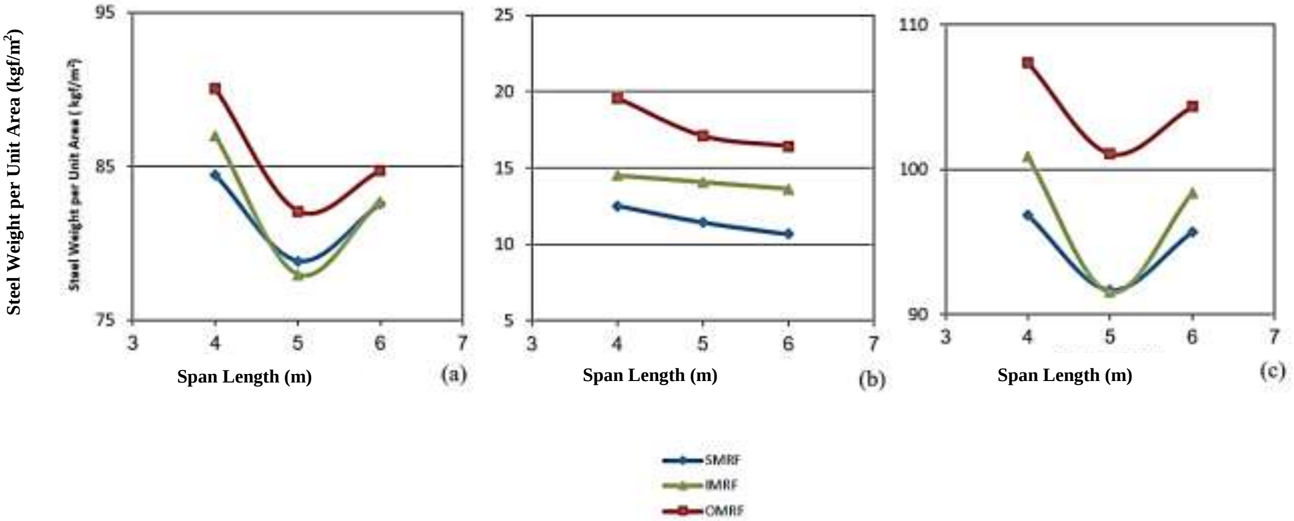


Fig. 6. Required steel material weight versus span length, (a) Beams and columns, (b) Connections, (c) The total amount (i.e. beams, columns and connections)

According to Figure 6, in the structures with the same number of stories, moving from the span length of 4 m to 5 m and then from 5 m to 6 m, the required steel weight initially decreases and then increases, which indicates that the span length of 5 m is the optimum length in the studied structures. This means that excessive short and long span lengths are not suitable for design and an optimum span length must be taken for which the designed profiles that bear the gravity load prepare the required stiffness to control the seismic loads and inter-story drifts. This may be interpreted such that this span length is the one which satisfies the criteria on strength and drift simultaneously. The results from IMRFs and SMRFs with span lengths of 5 m are quite close; it may be discussed that, although the base shears of SMRFs are generally lower compared to IMRFs, the drift limits are lower too, which is an unfavorable difference of SMRFs from IMRFs in contrast to their base shears. Thus, the required materials for SMRF and IMRF designs become very similar to each other.

In the following, the minimum and maximum amounts of required steel material weight per unit area for the designed beams and columns, connections and overall structures are given in Tables 2 and 3. It should be mentioned that the values given are calculated when no braces are used to control the story drifts. Use of braces, if practical with respect to the architectural plan, can dramatically improve the results (these numbers can be considered as upper and lower limits for rough check of design results in accordance with the amount of steel per unit area obtained from the design. This means that if the maximum or minimum designed values are, respectively, higher or lower than the limits given in Tables 2 and 3, it might be considered that the designed structure is over-designed or under-designed).

In the first three rows of Table 2, the maximum amounts of the total steel weights per unit area, in the second three rows the total steel weights for beams and columns and in the third three rows the required steel weights per unit area associated with the connections are given. The maximum total steel weight can be used as a factor to distinguish over-designed steel structures. For the present case, these values are 122 kg/m^2 for the 15-story and 89 kg/m^2 for the 5-story SMRFs, 131 kg/m^2 for the 15-story and 94 kg/m^2 for the 5-story IMRFs and 146 kg/m^2 for the 15-story and 101 kg/m^2 for the 5-story OMRFs. A higher design-based required total steel weight per unit area of a steel structure than these given values means that the structure is over-designed. In addition, it should be noted that the maximum weights for connections are 29 kg/m^2 for the 15-story and 11.5 kg/m^2 for the 5-story SMRFs, 29 kg/m^2 for the 15-story and 33 kg/m^2 for the 5-

story IMRFs and 37 kg/m^2 for the 15-story and 39 kg/m^2 for the 5-story OMRFs. These values are approximately from 22% to 25% total steel weight and constitute a considerable portion of structures' steel material weight.

In the first three rows of Table 3, the minimum amounts of the total steel weights per unit area, in the second three rows the minimum steel weights for beams and columns and in the third three rows the minimum steel weights for the connections are presented. The minimum total steel weights can be used as a factor to verify whether all according AISC 341-16 regulations are met in the design procedure. These values are 103 kg/m^2 for the 15-story and 68 kg/m^2 for the 5-story SMRFs, 105 kg/m^2 for the 15-story and 70 kg/m^2 for the 5-story IMRFs and 112 kg/m^2 for the 15-story and 74 kg/m^2 for the 5-story OMRFs. A lower design-based required total steel weight per unit area of a steel structure than these given values means that the intended structure is under-designed and the required steel shall be more than design values. Also, noteworthy is the fact that the minimum weights for connections are 10 kg/m^2 for the 15-story and 7 kg/m^2 for the 5-story SMRFs, 10 kg/m^2 for the 15-story and 10 kg/m^2 for the 5-story IMRFs and 16 kg/m^2 for the 15-story and 12 kg/m^2 for the 5-story OMRFs. These values are approximately 10% to 15% of the total steel weight.

Table 2
Maximum values of required steel weight per unit area

		Maximum values of steel weight per area (kgf/m^2) versus:							
		Number of Stories			**Span length (m)**			**Number of Bays**	
		15	9	5	6	5	4	6	3
All Members	OMRF	146	120	101	132	135	146	118	146
	IMRF	131	108	94	122	127	131	110	131
	SMRF	122	108	89	121	122	122	105	122
Beams & Columns	OMRF	116	97	78	116	114	116	99	116
	IMRF	109	93	77	103	107	109	99	109
	SMRF	109	92	77	107	109	104	92	109
Connections	OMRF	37	48	39	22	21	36	20	34
	IMRF	29	38	33	16	28	24	16	28
	SMRF	29	15	11.5	10.8	11.9	12.5	11.2	13.1

Table 3
Minimum value of steel weight per unit area

Table 3
Minimum value of steel weight per area (kgf/m^2)

		Number of Stories			**Span Length (m)**			**Number of Bays**	
		15	**9**	**5**	**6**	**5**	**4**	**6**	**3**
All Members	**OMRF**	**112**	**95**	**74**	**77**	**79**	**74**	**74**	**90**
	IMRF	**105**	**86**	**70**	**70**	**75**	**74**	**70**	**87**
	SMRF	**103**	**87**	**68**	**70**	**70**	**68**	**68**	**83**
Beams & Columns	**OMRF**	**95**	**76**	**62**	**65**	**65**	**62**	**62**	**73**
	IMRF	**89**	**74**	**60**	**60**	**64**	**64**	**60**	**73**
	SMRF	**90**	**61**	**61**	**62**	**61**	**61**	**61**	**73**
Connections	**OMRF**	**16**	**16**	**12**	**12**	**14**	**12**	**12**	**14**
	IMRF	**10**	**12**	**10**	**10**	**11**	**10**	**10**	**10**
	SMRF	**10**	**10**	**7**	**8**	**9**	**7**	**7**	**10**

4. Behavioral Comparison of Special, Intermediate and Ordinary Steel MRFs

4.1. Story Drifts

In this section a comparison regarding drift, force and stress distribution along the height of the structures is conducted. To fulfill this aim, six 3D structures with 15 stories and six 5-meter bays were designed. In the first group, there were no braces while in the second group the bottom stories were braced. In Figure 7, inter-story drifts of the special, intermediate and ordinary MRFs with and without braces in bottom stories are presented. According to the figure, the distribution of inter-story drifts in the SMRFs is more uniform and smoother due to more regular distribution of stiffness in the height of the frames; therefore, the probability of existence of a soft story in SMRFs is less than IMRFs and OMRFs. The observed dramatic decrease in the second story is the result of application of very strong girders in the first story (which can be considered as a base for the second story) which are used to control the drifts of the bottom story. This is while using similar braces in the three groups of structures can lead to significant decrease in the size of girders and a relatively smooth drift distribution.

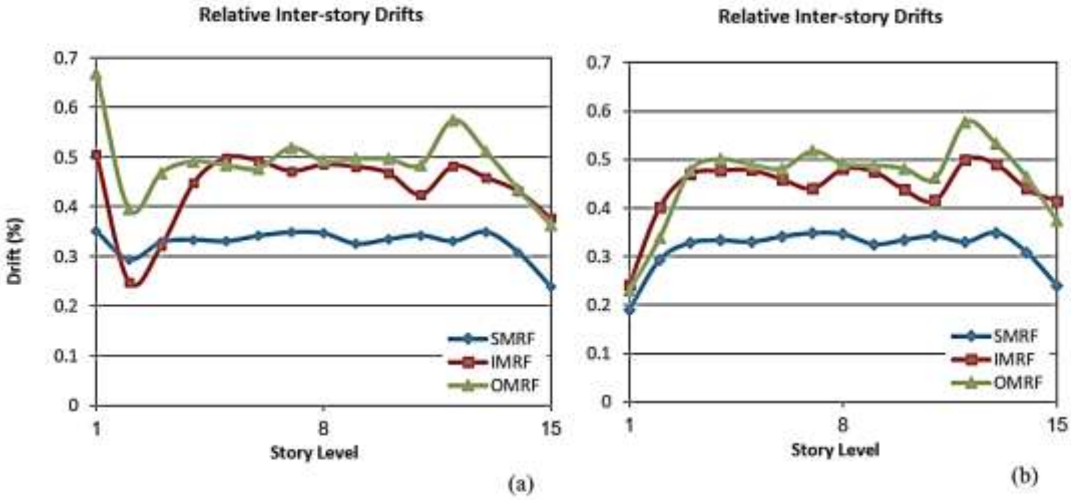


Fig.7. Story drifts of regular structures with six bays and 5-meter spans, (a) No braces, (b) Bottom stories braced

4.2. Lateral Displacements

In order to compare the lateral displacements of the MRFs, six 9-story 3D structures with fives bays were designed. Three structures of the first group were regular and their span lengths were all 5 m, while in the second group three structures were irregular and the span lengths were 7, 7, 4, 5 and 7 meters, respectively from the first span to the last.

Figure 8 depicts the drifts of the special, intermediate and ordinary MRFs. According to the figure, lateral displacements of the SMRFs are less than those of the IMRFs and OMRFs; hence, the separation joint (also called moat) in the SMRFs can be considered smaller, which can be rather effective and helpful in case of shortage of space.

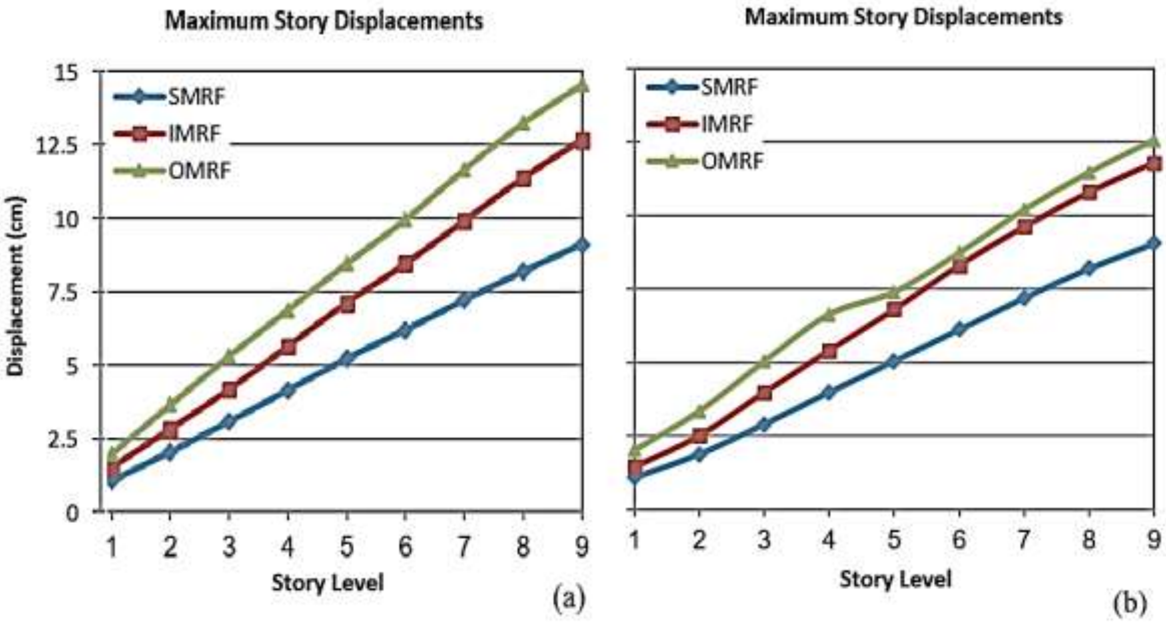


Fig 8. Maximum story displacements of 9-story structures with 5 bays (a) Uniform spans, (b) Non-uniform spans

The aforementioned characteristics are not just limited to regular structures, as it is clearly observed in Figure 8.b that this trend is similar in irregular structures as well.

4.3. Stress Distribution

For investigation of the stress distribution patterns in the frames, an average amount of stress is taken as a representative of stress in the beams and columns. Since the structures are regular, beam and column stresses of each story are almost equal. The maximum amount of stresses in any story for beams and columns are shown in Figure 9. It is observed that the distribution of stresses in the SMRFs is more uniform than in the IMRFs and SMRFs. In addition, regarding the maximum values of average stresses in the columns and beams of the SMRFs which equal 63% and 74% respectively, the weak beam-strong column requirement of the design code has been satisfied, which has resulted in columns with moderate stress ratios while the beams bear more stress ratios.

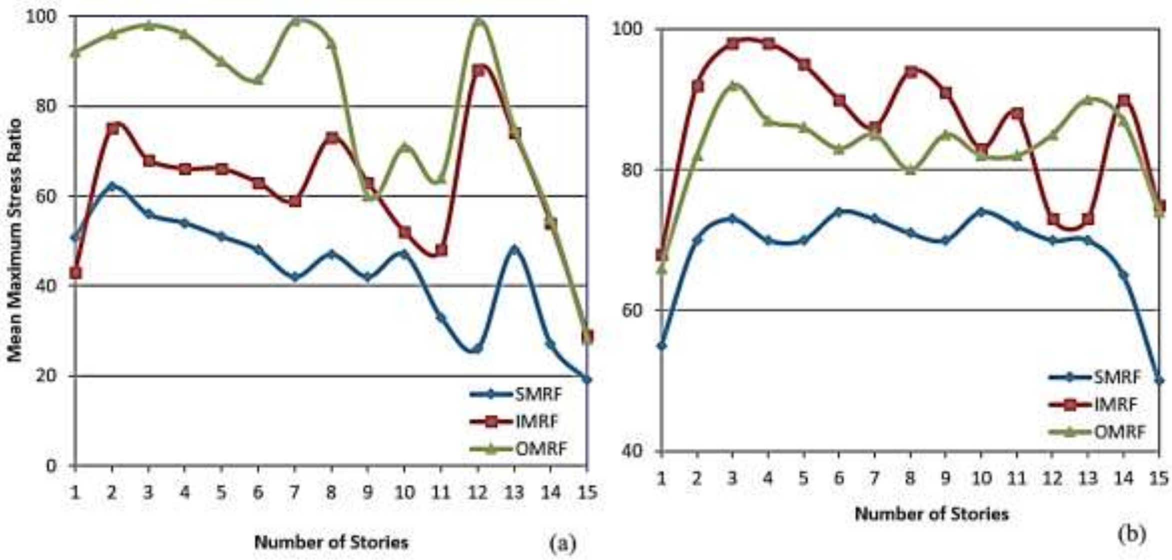


Fig. 9. Mean maximum stress ratios of 9-story structure with five 6-meter bays: (a) Columns, (b) Beams

5. Conclusions

The conformity of the moment resisting frame systems designs based on AISC-LRFD provisions, was assessed in this paper. Accounting for the recommended ductility and rotations in moment connections is a critical issue that needs to be further highlighted in standard procedures of structural design. The present study showed that following the recommended procedures in available standards results in reduction of the consumed steel in special moment resisting frames.

The average consumed steels per unit area for different MRFs in compliance to AISC 341-16 and FEMA 350 were estimated numerically. It is evident that the results are directly related to dead and live loads, structure's importance factor, soil type, seismicity of the zone and structure's dimensions. Considering these factors, 45 structures were designed in three categorizations, namely SMRF, IMRF and OMRF. These groups included structures with 3 and 6 bays and 5, 9 and 15 stories. The bay span lengths were taken 4, 5 and 6 meters. The structures were designed by implementing AISC 341-16 regulations. The results obtained from the simulations were compared from the economic and behavioral standpoints. The following conclusions can be drawn from the present study:

- While design of special connections and panel zones of SMRFs is more complicated and potentially more expensive, the construction cost in relation with the material needed for the elements and connections of these structures is lower compared to IMRFs and OMRFs.
- The governing design criterion in SMRFs and IMRFs is relative inter-story drift, while in OMRFs, strength is the governing design criterion.
- The design-based steel weight per unit area for columns in SMRFs is less compared to that in IMRFs and OMRFs. This is while design-based steel weight per unit area for beams in SMRFs is more compared to that IMRFs and OMRFs.
- The average total steel weight per unit area in SMRFs is 2 to 6 percent less than that in IMRFs.
- The average total steel weight per unit area in SMRFs is 8 to12 percent less than that in OMRFs.
- The average steel weight per unit area required for connections in SMRFs is 6 to 33 percent less than that in IMRFs.
- The average steel weight per unit area required for beams and columns in SMRFs is 44 to 48 percent less than that in OMRFs.
- The average steel weight per unit area required for beams and columns in SMRFs is 1 to 3 percent less than that in IMRFs.
- The average steel weight per unit area required for connections in SMRFs is 2 to 8 percent less than that in OMRFs.
- The maximum total steel weight per unit area in SMRFs is 1 to 4 percent less than that in IMRFs.
- The maximum total steel weight per unit area in SMRFs is 10 to 16 percent less than that in OMRFs.

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