AUT Journal of Civil Engineering

AUT J. Civil Eng., 1(2) (2017) 177-188
DOI: 10.22060/ceej.2017.12406.5207

An Overview of the Effects of High-Strength Reinforcement (HSR) on the Intermediate Moment-Resisting Frames

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ABSTRACT: Increasing volume of constructions and necessity of having economic structures lead to the production of high-strength reinforcement (HSR). HSRs have many benefits; however, because of limitation in producing ductile HSR and the effect of HSR in the reduction of overall ductility of reinforced concrete structures, its application has been limited in seismic prone areas especially in special Reinforced concrete(RC) moment-resisting frames. In this research, the effect of HSR application on drifts, displacements, and quantity of consumed steel are studied by the linear static analysis, and also the base shear and proportionate displacements to them are studied by the nonlinear static analysis (Pushover) with ETABS software (for nine models with different numbers of stories and grades of steel). Then, the tensile strains of beams' ends which can be a representation of cracking phenomenon in concrete are acquired conducting nonlinear dynamic analysis with Opensees. Ultimately, it is shown that although HSRs have economic benefits, they increase displacements and drifts. To compensate this issue, it is necessary to increase the rigidity of members by increasing steel quantity or dimension of members. This is a serious challenge because it neutralizes steel consumption reduction. It is also shown that by substituting the reinforcement bars for higher grade ones: the level of tensile stress in concrete alongside with the tolerated displacement in order to enter the nonlinear stage in Pushover analysis increases. Moreover, the less the grades of steel, the fewer shears are tolerated by structures.

Review History:

Received: 14 January 2017
Revised: 4 December 2017
Accepted: 11 December 2017
Available Online: 15 December 2017

Keywords:

High-strength reinforcement (HSR)
Displacement
Pushover
Non-linear dynamic analysis
Strain

1- Introduction

Over several decades the design of reinforced concrete structures was dominated by the application of steel reinforcement bars with yield strength (fy), equal to 280 MPa (40 ksi). In the late 1960s, the typical yield strength increased to f_y=420 MPa (60 ksi). A design with steel having higher yield strength values has been permitted; the 1971 edition of ACI 318 (1971), for instance, limited the yield strength to $f_y \leq 560$ MPa (80 ksi), Lepage et al. 2008 [1]. As another example, ACI 318-08 permits the application of reinforcement bar having a design yield strength, defined as the stress corresponding to a strain of 0.0035, not exceeding 560 MPa (80 ksi) [2]. With more development in technology, the capability of producing higher grade reinforcements was reached and the temptation of constructing with less amount of materials such as steel and concrete made researchers focus on the application of stronger materials.

Using high-strength reinforcements have some advantages: decreasing the number of laborers and material expenses, expenses of peripheral issues such as cranes or transportation and overhead expenses, (Kheyroddin and Arshadi [3, 4]), decreasing construction time duration and making the process of construction easier. The expenses of reinforcements are nearly 30 percent of the structural expenses, and reduction of their amount can make constructions more economical. However, application of them has also some disadvantages such as: 1) The crack width under service and design loads, because the elasticity module of high strength reinforcements are similar to the ordinary ones and their levels of tensions are higher than theirs, then their crack width under service and design loads will be larger, 2) The brittle failure threat in which concrete will be crashed before steel yielding, for this reason, the codes limit the yielding stress of steels, 3) The ambiguous effect of them on seismic behavior of the structures limits their application in areas with a high hazard of seismicity [5]. These effects are the subject of many pieces of analytical and experimental research in these days.

AASHTO LRFD Specifications (AASHTO 2007) also limit the use of reinforcing yield strength in designing to at least 420 MPa (60 ksi) and no greater than 525 MPa (75 ksi) [6]. Thus, AASHTO prevents the application of 280 MPa (75 ksi) reinforcing steel whereas ACI continues to allow its use. Both ACI and AASHTO limits have been written and interpreted not to exclude the use of higher strength grades of steel, but only limit the value of fy that may be used in the design calculations. The limits on yield strength are primarily related to the prescribed limit on the concrete compressive strain of 0.003 and to the control of crack widths at service loads. Crack width is a function of steel strain and consequently steel stress (Nawy 1968) [7]. Therefore, the stress in the steel reinforcement needs to be limited to some extent to prevent cracking from affecting the serviceability of the structure.

Due to the importance of the application of HSRs, many researchers have been studied different aspects of their application. The cyclic response of concrete beams reinforced with high-strength steel reinforcements was studied by Tavallali [8]. Restrepo et al. [9] tested two circular cantilever bridge columns. The results give a positive indication that high-strength reinforcement can be used successfully up to drift levels approaching 4%. They tested concrete columns reinforced with grade 120 longitudinal bars and compared the results to tests with grade 60 bars.

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The column test data indicated that replacing conventional grade 60 longitudinal reinforcement with reduced amounts of grade 120 reinforcement leads to comparable flexural strength and deformation capacity. Shahrooz et al. [10] researched comprehensively about the bond and serviceability characterization of concrete reinforced with high strength steel. Kheyroddin and Arshadi also overviewed the seismic consideration of the application of HSRs in earthquake resisting structures [3]. They also investigated the application of HSRs in special moment resisting frames in technical materials [4]. Kheyroddin and Mortezaee have also investigated the plastic hinge characteristics in RC frames [11].

This study aims to overview the effect of using high-strength reinforcement bars on the drifts, displacements, base shears and the number of consumed reinforcements by linear and non-linear static analysis with the ETABS software (Wilson & Habibullah). The non-linear behavior of elements is considered by assigning displacement and force control hinges on them. Then the strains of both ends of the beams of structures are acquired with the non-linear dynamic analysis with Opensees. In this case, the non-linear behavior is applied by using a distributed plasticity.

The results showed that, although the number of reinforcements decreases by using HSR, the drifts and displacements increase. This is a paradoxical challenge that needed to be compensated by adding more steel. This means neutralizing the benefit of reducing steel consumption. It is also shown that by substituting reinforcement bars for higher grade ones: the level of tensile stress in concrete alongside with the tolerated displacement in order to enter the non-linear stage in pushover, increases. Moreover, the less the grade of steel, the fewer shears are tolerated by structures.

2- Methodology and Modelling

In this paper, three different concrete structures with intermediate moment-resisting frame systems (3, 7 and 10-story structures) with three different grades of steel (grade 40, 60 and 80 reinforcements) are modeled. The height of each story is 3.2 meters, then the overall height of the structures are 9.6, 22.4 and 32 meters, respectively which do not exceed the 35 meter limit for the overall height of intermediate moment-resisting frames in the Iranian Code of Practice for Seismic Resistant Design of Buildings (BHRC 2015-the fourth edition) [12]. The plans of all structures are the same and symmetric which are shown in Figure 1. All of the structures have 4 spans with the length of 6 meters.

Moment-resisting frames tolerate a lateral load by rotation of joints, shear forces and moments generated in beams. Although turnover moment (induced by lateral loads) produces axial forces in columns, because of the rigidity of slabs, the axial forces in beams are disregarded. It is worth saying that all the floors are regarded rigid and to remain elastic which ends up the same displacement for all the frames and disregarding axial forces in the beams.

Based on the mentioned code, it is assumed that the structures are located in the area with type 3 soil and with the very high level of seismicity risk (A=0.35 g). Also, the importance factor is assumed I =1. Then according to BHRC 2015 [12] the design time period of structures which are the minimum amount of analytical time period and 1.25 times of experimental time period, are obtained. Furthermore, in this study R (response modification factor) is considered equal to 5 (the lateral resisting system is regarded as the intermediate moment-resisting frames). Finally, the earthquake coefficient (C) of these structures and time periods are calculated as shown in Table 1.

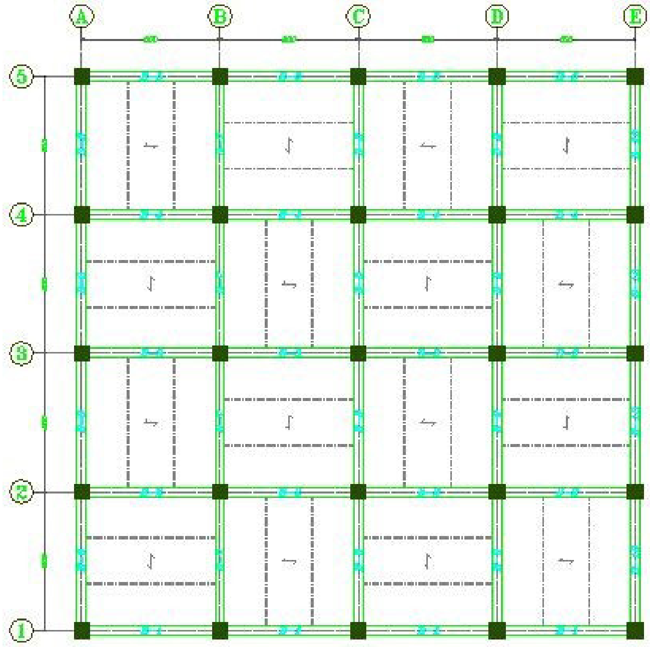

Fig. 1. The plan of structures

$$C = \frac{ABI}{R} \qquad (1)$$

Table 1. Seismic characteristics of models

Models	Overall height (m)	Time period (s)	C
3-storey model	9.6	0.48	0.165
7-storey model	22.4	1.03	0.1204
10-storey model	32	1.41	0.094

The compressive strength of concrete used in the models is considered 25 (kN/m^2) and each of the models is designed by three different grades of reinforcements with 280, 420 and 560 MPa as yield strength (grade 40, 60 and 80 ksi) separately to compare the effect of yielding strength. The ultimate and yield strengths of these grades are shown in Table 2. Gravity loads consist of 3.1 (kN/m^2) as the dead load and 1.5 (kN/m^2) as the live load for the roof, and for all the other floors: 3 (kN/m^2) as the dead load and 2 (kN/m^2) as the live load.

Table 2. Seismic characteristics of models

Grades	Yield strength (MPa)	Ultimate strength (MPa)
40	280	420
60	420	630
80	560	790

Firstly, without consideration of the cracking effect, models are designed by the ETABS software (Wilson & Habibullah) to calculate the volume of reinforcements of each structure for grade 40 reinforcement bars. Then by assuming the fact that the dimension of concrete members and loading situation are the same, the equivalent amount of bars in different cases of using other grades, are calculated by Equation 2:

$$N_1 A_{s1} f_{y1} = N_2 A_{s2} f_{y2} \tag{2}$$

In which, N is the number of reinforcement bars, f_y is the yield stress and A_s is the area of bars. It is worth mentioning that because of not yielding bars in columns (in comparison to the bars of beams), this method of finding an equivalent number of bars has less accuracy for columns. This phenomenon is due to the strong column-weak beam principle which is considered in designing.

By computing the area and percentage of consumed steel, the effective moment inertia of columns and beams are obtained by using the formula below (ACI 318-14) [13].

Beam:

$$0.25 I_g \le \left(0.1 + 25p\right)\left(1.2 - 0.2\frac{bw}{d}\right) I_g \le 0.5\, I_g \tag{3}$$

Column:

$$0.35 I_g \le I = \left(0.8 + 25\frac{Ast}{Ag}\right)\left(1 - \frac{Mu}{Puh} - 0.5\frac{Pu}{P0}\right) I_g \le 0.875 \tag{4}$$

Where: I_g is the gross inertia moment of the member, p is the percentage of bars, d is the effective depth of the beam, b_w is the width of the web of the beam, h is the height of column section, A_g is the gross section area of member, A_{st} is the steel area of cross-section, M_u is Applied moment, P_u is Applied axial force and P is Axial capacity of the column.

The ratio of effective inertia moment of beam and columns to the gross ones are obtained by the above formulations for each grade that are shown in Table 3.

Table 3. The ratio of effective inertia moment of beam and columns to the gross ones for each grade

Grades	Beam	Column
40	0.45	0.81
60	0.35	0.7
80	0.3	0.65

It is worth saying that to explore the influence of yield strength of steels on displacements, drifts and the amount of consumed steel, in case of using each grade, the dimension of column and beam sections are considered the same for each structure with the same number of stories. The dimension of beam and columns, the number and the size of bars in all the models are shown in Tables 4 to 6.

The reason behind making the beams and columns typical is to decrease the error in the modeling process in Opensees software.

Ultimately, in order to calculate the drifts and displacements with the linear static analysis and performing non-linear static analysis (pushover), the ETABS software is used and the effective inertia moment of members calculated by the above formula are considered in both analyses. As for calculations of strains in the endpoints of beams with non-linear dynamic analysis, the Opensees software is used. The structures are designed based on ACI318-14 and the parameters of pushover analysis are obtained based on ASCE41-17 [14].

In order to verify the two mentioned software (Opensees and ETABS) and calibration, the concrete frame tested by Hemmati et al. [15] is taken into consideration. The geometry of Hemmati's frame is shown in Figure 2.

Table 4. The characteristics of three-story structures

Three-story structure										
Story	Column dimension (cm)		Beam dimension (cm)		280 (MPa)		420 (MPa)		560 (MPa)	
	Height	Width	Height	Width	Number of bars	Diameter of bars (mm)	Number of bars	Diameter of bars (mm)	Number of bars	Diameter of bars (mm)
1	55	55	40	40	12	28	12	25	12	22
2	45	45	40	40	12	25	12	22	12	20
3	40	40	40	40	12	22	12	20	12	18

Table 5. The characteristics of seven-story structures

Seven-story structure										
Story	Column dimension (cm)		Beam dimension (cm)		280 (MPa)		420 (MPa)		560 (MPa)	
	Height	Width	Height	Width	Number of bars	Diameter of bars (mm)	Number of bars	Diameter of bars (mm)	Number of bars	Diameter of bars (mm)
1	70	70	60	50	20	28	20	25	20	25
2	60	60	60	50	20	25	16	25	16	25
3	60	60	60	50	16	20	16	20	16	20
4	60	60	50	50	16	20	16	20	16	20
5	50	50	50	50	12	20	12	20	12	20
6	45	45	40	40	12	20	12	20	12	20
7	45	45	40	40	12	20	12	20	12	18

Table 6. The characteristics of ten-story structures

Ten-story structure										
Story	Column dimension (cm)		Beam dimension (cm)		280 (MPa)		420 (MPa)		560 (MPa)	
	Height	Width	Height	Width	Number of bars	Diameter of bars (mm)	Number of bars	Diameter of bars (mm)	Number of bars	Diameter of bars (mm)
1	95	95	70	50	24	32	24	28	24	26
2	90	90	70	50	20	28	20	25	20	25
3	80	80	70	50	20	28	20	22	20	22
4	70	70	60	50	20	28	20	22	20	20
5	70	70	60	50	20	25	20	20	20	20
6	60	60	60	50	16	25	16	20	16	20
7	60	60	50	50	16	22	16	20	16	20
8	50	50	50	50	16	22	12	20	16	20
9	50	50	40	40	12	22	12	20	12	20
10	40	40	40	40	12	22	12	20	12	18

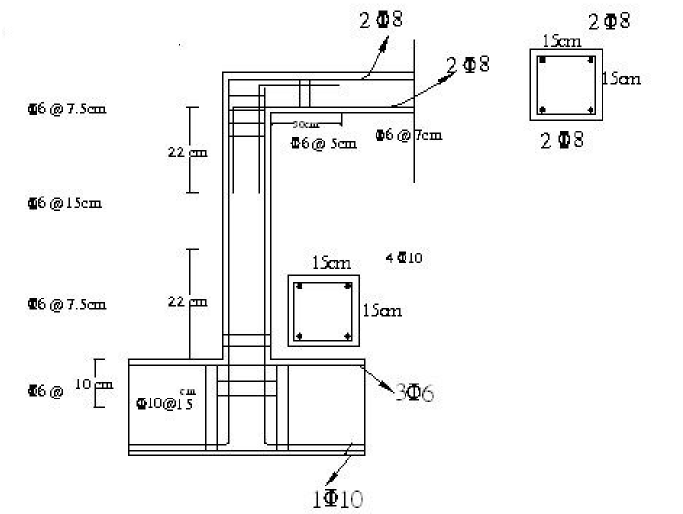

Fig. 2. The geometry of concrete frame [15].

3- Results and Discussion

In this research, the drifts and displacements of stories with linear-static analysis (by ETABS software) and the amount of consumed steel with Saze90 software are achieved for nine models and compared to each other. Then the non-linear-static analysis (pushover) is performed to explore the effect of the application of high-strength reinforcement on the base shear and displacements (with regard to the non-linear behavior of members). Finally, it is tried to compare the tensile strains of both ends of the beams by using non-linear dynamic analysis with Opensees software, which can be a representation of cracking phenomenon in the beams.

3- 1- Displacements and Drifts

As discussed in the previous paragraphs, by using high-strength reinforcements (HSR), on the one hand, the stress level in steels is increased which leads to increasing the cracks and decreasing the stiffness and rigidity of concrete members [5]. Then, with the formation of more cracks and decreasing the stiffness, the drifts and displacements increase which can be a negative issue. Drifts and displacements should be limited because of non-structural considerations. On the other hand, the amount of the consumed steel decreases by using high-strength bars which is cost effective and positive phenomenon (as discussed in the introduction part). This is a paradoxical challenge: on the one hand, with the application of high-strength reinforcements (HSR), drifts increase and to overcome this problem the rigidity of members should be enhanced by increasing either the dimension of members or the amount of steel. This destroys the essential benefit of high-strength reinforcements (HSRs).

Figure 3 shows the displacements of floors in the three-story structure. The difference of displacements of the first, second and third floors in case of using grades 40 and 60 are 22.5, 23.7 and 25.2 percent respectively and the average of them is 23.8. As for using grade 80 instead of grade 60, the displacement of floors increases 10.5, 11.5 and 12.7 percent, respectively with 11.57 percent as the average amount. The maximum amount of the increase in drifts and displacements belong to the roof story. This phenomenon was expected because by using HSRs and cracking phenomenon, the member's stiffnesses are reduced and accordingly the time period and softness of structures increase. This ends up increasing responses of the structures such as displacements and drifts. Also, by increasing the grade of steel more and more, the intensity of increasing displacements decreases. This can be justified by the formulation shown above for the effective inertia moment of beams and columns. However, these formulations need to be verified by experimental results.

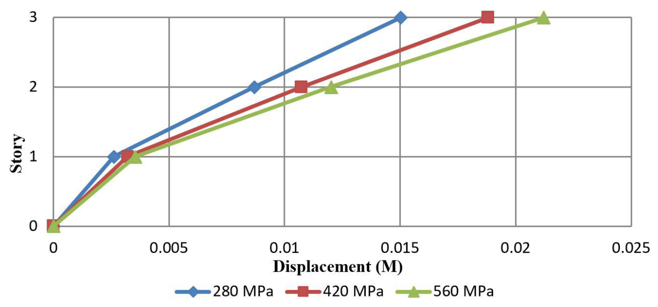

Fig. 3. A comparison of displacements of floors with different grades in the three-story structure.

Figure 4 displays the comparison between drifts of floors with different grades in the three-story structure, including grade 40, 60 and 80 reinforcement bars. By using grade 60 instead of grade 40, the drifts of floors increase 22.5, 24.4 and 27.1 respectively, whose average is 24.7. Moreover, by substituting grade 60 for grade 80, the increase in the drift percent is 10.4, 12 and 14.4 percent whose average is 12.3. These drifts have a similar influence under using HSRs with displacements.

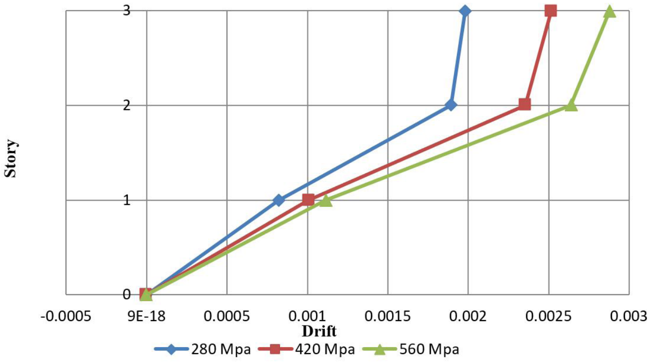

Fig. 4. A comparison of drifts of floors with different grades in the three-story structure.

As shown in figures below this process is repeated for the other models as well. Figures 5 to 8 display the comparison of the drifts and displacements of floors with different grades in seven and ten-story structures. Tables 7 to 9 show the increase percent of drifts and displacements in three, seven and ten-story structures.

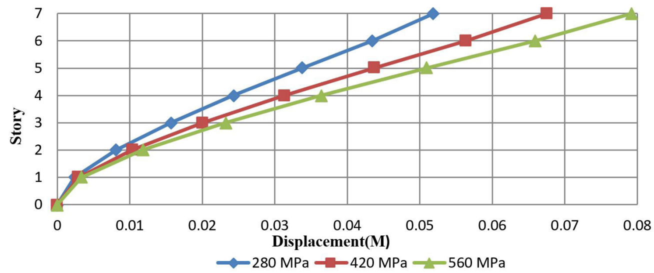

Fig. 5. A comparison of displacements of floors with different grades in the seven-story structure.

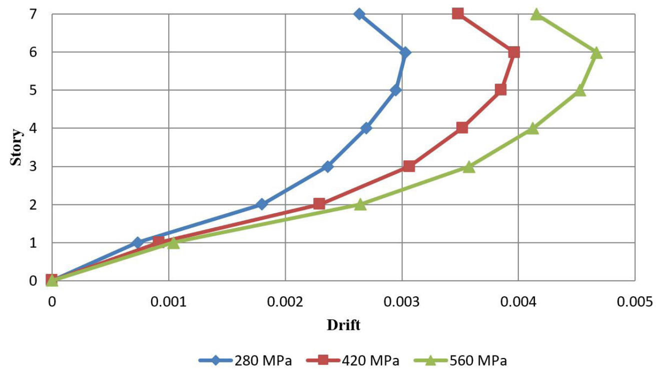

Fig. 6. A comparison of drifts of floors with different grades in the seven-story structure.

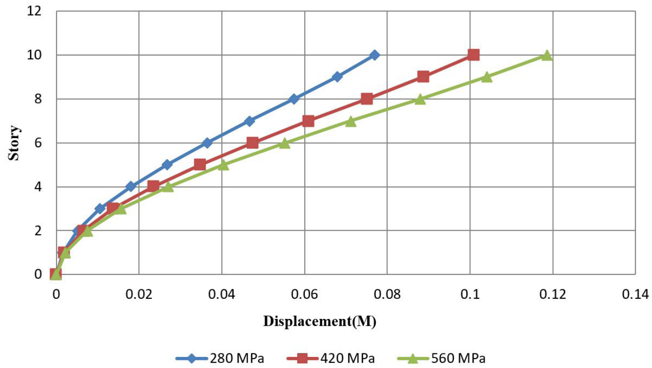

Fig. 7. A comparison of displacements of floors with different grades in the ten-story structure.

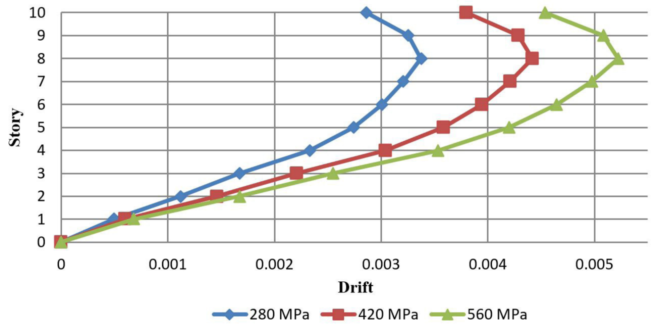

Fig. 8. A comparison of drifts of floors with different grades in the ten-story structure.

It is worth mentioning that all of the drifts are checked and are less than the allowable design drift amount which can be calculated by the formulation mentioned in the Iranian Code of Practice for Seismic Resistant Design of Buildings (BHRC 2015-the fourth edition) [12]. However, if they did not accord with allowable amounts, it would be essential to add more reinforcements or increase the dimension of member sections to enhance their inertia moment and the stiffness of the structures. This would be crucial because it neutralizes the steel consumption reduction.

$$\text{Drift} = \begin{cases} \leq \dfrac{0.025}{Cd} & \text{For the structures with less than 5 floors} \\ \leq \dfrac{0.02}{Cd} & \text{For the structures with more than 5 floors} \end{cases} \tag{5}$$

Where C_d is the coefficient of non-linear displacement (amplification factor of displacement) and for intermediate moment-resisting frames are 4.5.

3- 2- Amount of Consumed Steel

As discussed above, the amount of steel by using high-strength reinforcements (HSR) decreases which leads to making the construction economical and easier. This also can prevent the congestion of bars in the members and make the process of pouring concrete into the cast easier and with fewer problems. In this part, it is tried to calculate the reduction percentage of consumed steel in the beams of all structures by using HSRs. The volume of steel in beams is calculated with regard to the detail drawing obtained by using the Saze90 software. Figure 9 shows the comparison of steel amount in the three-story structure. It is shown that by substituting grade 60 for grade

Table 7. The displacement and drift increase percent of three-story structures

Story	Substitution grade 60 with 40		Substitution grade 80 with 60	
	Displacement increase percent	Drift increase percent	Displacement increase percent	Drift increase percent
1	22.5	22.5	10.5	10.4
2	23.7	24.4	11.5	12
3	25.2	27.1	12.7	14.4
Average	23.8	24.7	11.57	12.3

Table 8. The displacement and drift increase percent of seven-story structures

Story	Substitution grade 60 with 40		Substitution grade 80 with 60	
	Displacement increase percent	Drift increase percent	Displacement increase percent	Drift increase percent
1	25.3	25.4	13.1	13.1
2	26.9	27.6	14.5	15.1
3	28.3	29.8	15.6	16.8
4	29.1	30.5	16.2	17.4
5	29.5	30.6	16.6	17.6
6	29.8	30.7	16.9	17.8
7	30	32.2	17.2	19.1
Average	28.4	29.5	15.7	16.7

Table 9. The displacement and drift increase percent of ten-story structures

Story	Substitution grade 60 with 40		Substitution grade 80 with 60	
	Displacement increase percent	Drift increase percent	Displacement increase percent	Drift increase percent
1	22.1	22.1	12.9	12.8
2	28.1	30.5	13.8	14.1
3	29.9	31.8	14.6	15.4
4	30.2	30.5	15.3	16.3
5	30.4	30.7	15.9	17.1
6	30.5	31	16.4	17.8
7	30.6	31.2	16.8	18.2
8	30.7	30.9	17.1	18.2
9	30.8	31.8	17.3	18.5
10	31.1	32.7	17.5	19.4
Average	29.5	30.3	15.8	16.7

40, the volume of steel in beams decreases 23 percent and by replacing grade 80 with 60 this amount is 18 percent.

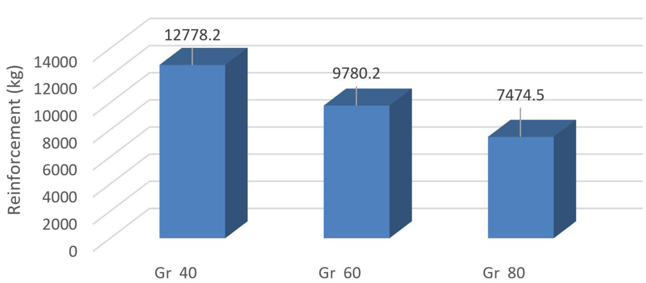

Fig. 9. Comparison between the amounts of consumed steel in the three-story structures

Figure 10 shows that in the seven-story structure, by substituting grade 60 for grade 40 the volume of steel in beams decreases 23 percent and by replacing grade 80 with 60, this amount is 24 percent.

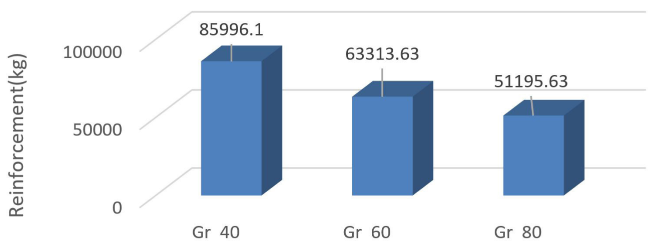

Fig. 10. A comparison between the amounts of consumed steel in the seven-story structures

Figure 11 indicates that in the ten-story structure, by substituting grade 60 for grade 40 the volume of steel in beams decreases 27 percent and by replacing grade 80 with 60 this amount is 19 percent. As it is clear, nearly 20 percent of construction expenditures are directly related to steel bars, which by using HSRs can be decreased significantly. This is along with the fact that decreasing amount of steel can: 1) decrease time duration of construction which leads to the reduction of overhead expenses, 2) decrease the peripheral expenses such as material transportation expenses or expenses of cranes, 3) facilitate the process of construction for laborers which reduces their expenditures and so on.

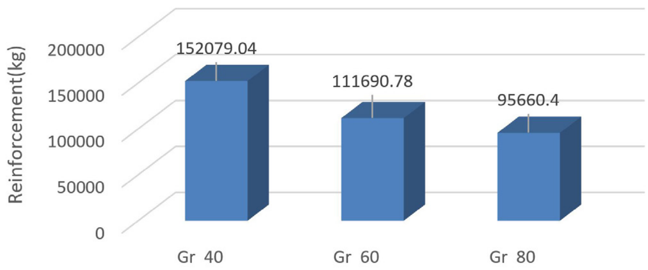

Fig. 11. A comparison between the amounts of consumed steel in the ten-story structures

3- 3- Pushover Analysis

In order to better comprehend the effect of HSR's application, the non-linear static analysis is performed by ETABS software and assigning the displacement and force control hinges to the members of frames manually. M3 displacement control hinges are assigned at both ends of the beams and P-M2-M3 displacement control hinges are assigned to both ends of columns. The mass center of the roof is considered as the control point and the target displacement is calculated based on the approximate relation mentioned in ASCE41-17 [14].

In the pushover analysis, monotonically increasing lateral forces are applied to a non-linear mathematical model of the building until the displacement of the control node at the roof level exceeds the target displacement. However, it is now well-known that these simplified procedures based on invariant load patterns are inadequate to predict inelastic seismic demands in buildings when modes higher than the first mode contribute to the response and inelastic effects alter the height-wise distribution of inertia force.

$$\delta_t = C_0 C_1 C_2 C_3 S_a (T_e^{\,2} / 4\pi^2) \tag{6}$$

where C_0=modification factor to relate the spectral displacement and expected maximum elastic displacement at the roof level; C_1=modification factor to relate the expected maximum inelastic displacements to displacements calculated for linear elastic response; C_2=modification factor to represent the effects of stiffness degradation, strength deterioration, and pinching on the maximum displacement response; C_3=modification factor to represent increased displacements due to dynamic second-order effects; T_e=effective fundamental period of the building in the direction under consideration calculated using the secant stiffness at a base shear force equal to 60% of the yield force; and Sa=response spectrum acceleration at the effective fundamental period and damping ratio of the building. The factors C_1, C_2, and C_3 serve to modify the relation between mean elastic and mean inelastic displacements where the inelastic displacements correspond to those of a bilinear elastic–plastic system. The effective stiffness, K_e, the elastic stiffness, K_i, and the secant stiffness at maximum displacement, K_s are identified in Figure 12. To calculate the effective stiffness, Ke, and yield strength, V_y, line segments on the force-displacement curve were located using an iterative procedure that approximately balanced the area above and below the curve.

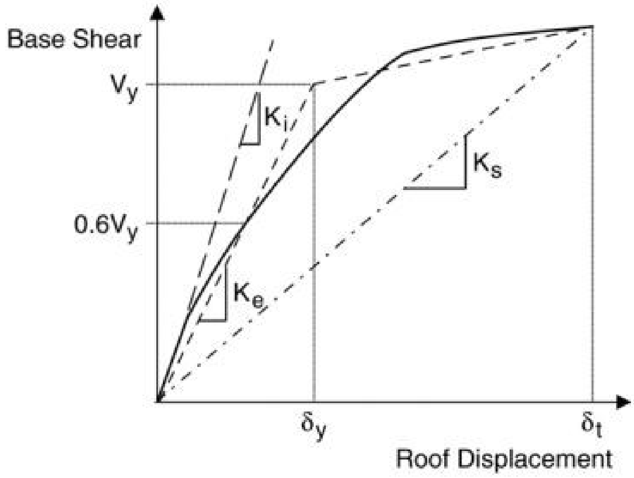

Fig. 12. Idealized force-displacement curves

The approximate amount of target displacement which is 1.5 times of the 0.02*h* can be considered for the target displacement (Where *h* is the overall height of the structure from the seismic base elevation). This amount is checked with another formulation recommended in ASCE41-17 [14].

Two sets of lateral load distributions are recommended in ASCE41-17 for non-linear static analysis. The first set consists of a vertical distribution proportional to (a) pseudo lateral load (this pattern becomes an inverted triangle for systems with fundamental period $T_1 < 0.5$ s); (b) elastic first mode shape; (c) story shear distribution computed via response spectrum analysis. The second set encompasses mass proportional uniform load pattern. In this research, the inverted triangle load pattern is used.

The force-displacement relationship of members asserted by non-linear relations is acquired by either experimental or analytical results. In this research, the idealized diagram of non-linear behavior manifested in ASCE41-17 [14] is used. Figure 13 shows this force-displacement relation.

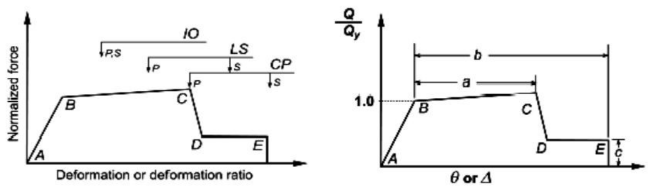

Fig. 13. The idealized diagram of nonlinear behavior

IO, LS, CP are acceptance criteria; θ and Δ are rotation and displacements (for more information it can be referred to ASCE41-17).

Figures 14 to 16 show the pushover diagram of the 3, 7 and 10-story buildings. With regard to these diagrams, the less amount of the yield strength, the more shear force is tolerated by the buildings. Moreover, by increasing the yield strength of the steel, more displacements are tolerated by structures before entering the non-linear stage. As for the formation of plastic hinges, the distribution of plastic hinges is located in the beams instead of columns. This can be justified by the principle design of strong column-weak beam. Moreover, although displacements are not affected by the strength of steel in the elastic domain, this phenomenon is not observed in pushover diagrams.

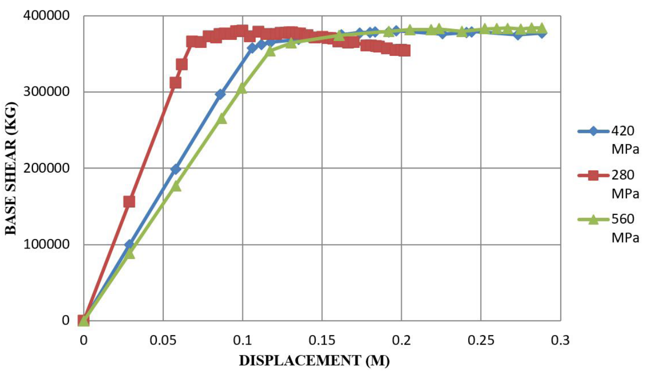

Fig. 14. comparison between the pushover diagrams in the three-story structures

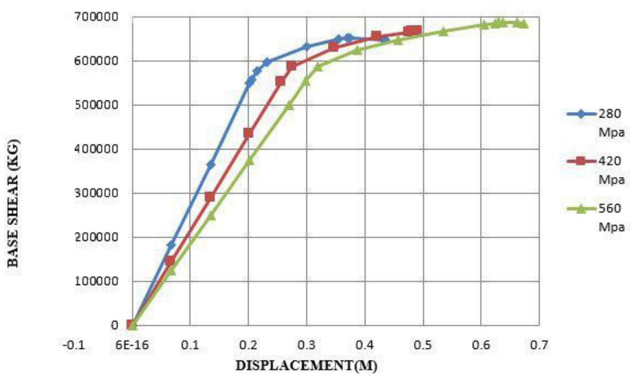

Fig. 15. A comparison between the pushover diagrams in the seven-story structures

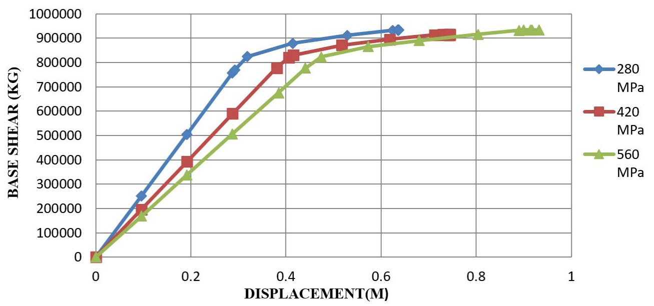

Fig. 16. A comparison between the pushover diagrams in the ten-story structures

3- 4- Strains

Stresses and strains of concrete can be significant criteria to investigate the effect of HSR application. As mentioned above, by using HSRs the level of tension is increased in concrete and as a result, the cracks develop in terms of number and width. Stresses and especially strains can be a representation of the tension level and crack in the concrete members. To investigate the effect of HSRs on strains, the two dimensional model of side frames of all nine models (3, 7, and 10-story structures with grade 40, 60 and 80 steel) are modeled in Opensees software based on what has been designed in ETABS Software before.

Analytical models were created using the open source finite element platform, OpenSees. Two-dimensional models of the side frames were developed for each building. A force-based non-linear beam-column element (utilizing a layered fiber section) was used to model all components of the frame models. Steel was modeled using a bilinear stress-strain curve with 2% post-yield hardening while the Kent-Park concrete model (which is an effective model due to experimental results [15]) in OpenSees was used to model the concrete section. Centerline dimensions were used in the element modeling, the composite action of floor slabs was not considered, and the columns were assumed to be fixed at the base level. For the time-history evaluations, masses were applied to frame models based on the floor tributary area and distributed proportionally to the floor nodes. Confined properties were generated using the well-known and widely-used Mander's confinement model. Plastic rotation in OpenSees is defined as the maximum absolute total rotation minus the yield or recoverable rotation.

For HSR, OpenSees software provides reinforcing steel material based on the uniaxial steel model proposed by Chang and Mander [17] and can consider the effect of plastic strain amplitude and buckling. Moreover, reinforcing steel material gives three parameters to reflect hysteretic laws incorporating degradation behavior with strain range and the number of cycles.

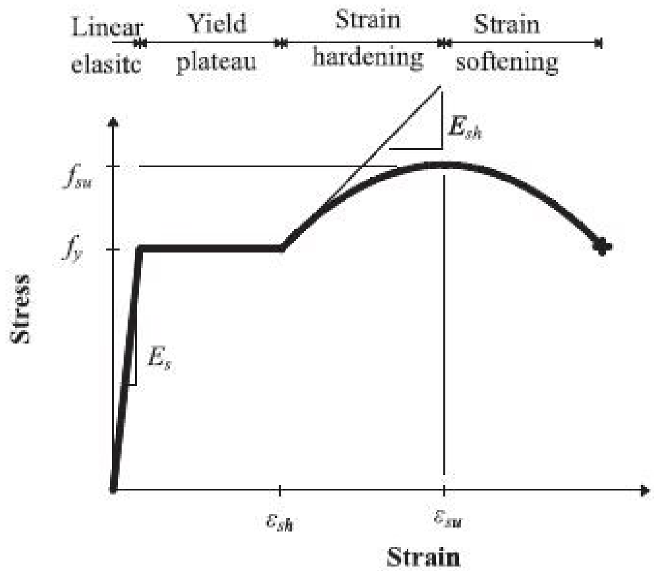

Fig. 17. Parameters of reinforcing steel material

Concrete 02 given by OpenSees software was used commonly to simulate unconfined or confined concrete. For skeleton curves, the modified Kent–Park model was used to describe the compressive behavior of un-confined or confined concrete [15]. The model proposed by Mohd Hisham [18] was used to express tensile behavior and reflect tensile damage. The parameters required are shown in Figure 18 (for more information about them, refer to Wang et al. [16]).

Firstly, each of the joint points is marked up as nodes and each element is marked up as elements. Secondly, under non-linear time-history analysis, the strains of the top and bottom nodes of each end of beams are calculated (as the positive and negative strains).

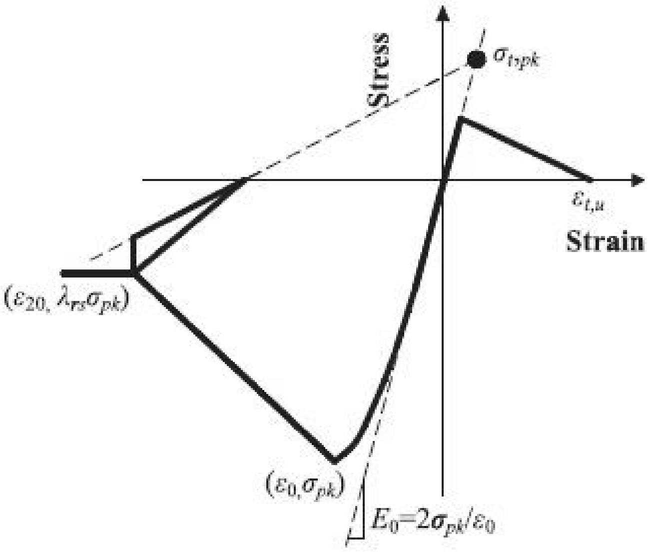

Fig. 18. Parameters of concrete 02

Seven pairs of ground motions were selected from the strong ground motion database of the Pacific Earthquake Engineering Research (PEER) Centre. The selected ground motions were far-field records and corresponded to locations which were at least 10 km from a rupturing fault. It is worth saying that among each pair of records, the records with a higher amount of PGA[1] were selected. The specifications of these records are shown in Table 10. The scaling of records is done according to the Iranian Code of Practice for Seismic Resistant Design of Buildings (BHRC 2015-the fourth edition) [12].

Positive strains which are tensile ones should be focused on because they cause the concrete cracking. It is worth saying that because seven records are used in this analysis, the average amounts of results should be considered as the ultimate results of the non-linear time-history analysis. As an example, the amount of strain increase the percentage of the critical beam which is the corner beam on the first floor in case of using different grades shown in Table 11.

Firstly, it is observed that by increasing the yield strength, the positive strains increase. As an example, in case of substitution grade 60 for 80 in three-story structures, the strain increase percentage is 7.53 or for substitution grade 40 for 60 is 10.35. It is also observed that the intensity of increasing strains by increasing the grade of steel, decreases. This can be justified by the fact that the difference of effective inertia moments of grade 40 and 60 steel based on the formulation mentioned in the section above are more than the ones with grade 60 and 80. It seems that there is a necessity to reconsider that formulation by more testing and experimental research.

Moreover, by increasing the number of stories, this process is expedited which means more and more cracking. This can be due to increasing the time period and ductility of structures and increasing the weight of structures by adding their height. The average of strain increase ratio of all beams by substituting steel for the higher grade ones is also shown in Table 12. Finally, it is worth mentioning that the number of ratios are also dependent on the specifications of records (such as magnitude, PGA and etc.). It means that by using higher scale factors or more intense records in the nonlinear time-history analysis, the number of ratios increases but the trends possibly remain the same.

4- Results

Increasing number of human population makes it necessary to construct more economically and rapidly. For this reason, researchers are trying to replace low strength materials with high strength materials to reduce their consumption. High-strength reinforcements (HSR) have many benefits, however, because of limitation in producing HSRs and the effect of HSRs in the reduction of overall structural ductility, their application has been limited in seismic prone areas. In this research, it is tried to research the effect of HSR application on drifts, displacements and quantity of consumed steel by static linear analysis, and also the base shear tolerated by structures calculated by non-linear static analysis with ETABS software (for nine models with a different number of stories and grades

Table 10. The specifications of seven records for nonlinear time-history analyses

Date	Earthquake name	Magnitude (Ms)	Station number	Significant duration (s)	PGA (g)	Abbreviation
1999/09/20	Chi-Chi, Taiwan	7.6	TCU047	13.11	0.413	Chi-Chi P1425
1992/04/25	Cape Mendocino	7.1	89324 Rio Dell Overpass-FF	15.34	0.385	Cape Mendicino P0810
1989/10/18	Loma Prieta	7.1	47006 Gilory – Gavilan Coll	5	0.357	Loma Prieta P0764
1966/06/28	Parkfield	6.1	1438 Temblor Pre-1969	5.1	0.357	Parkfeild P0034
1971/02/09	San Fernando	6.6	2478 Castaic – Old Ridge Route	14.5	0.324	San Fernando P0056
1980/06/09	Victoria, Mexico	6.4	6604 Cerro Prieto	8.6	0.621	Victoria, Mexico P0266
1987/10/01	Whittier Narrows	5.7	14403 LA-116th St School	6.58	0.396	Whittier Narrows P0630

Table 11. Strain increase ratio by using grade 40, 50 and 60 reinforcements for the critical beam

Number of stories	Strain increase percentage ratio	
	Substitution grade 60 with 40 (ksi)	Substitution grade 80 with 60 (ksi)
3	10.35	7.53
7	4.61	4.35
10	6.41	5.73

1 peak ground acceleration

Table 12. The average strain increase ratio of beams by using grade 40, 50 and 60 reinforcements

Number of stories	Strain increase percentage ratio	
	Substitution grade 60 with 40 (ksi)	Substitution grade 80 with 60 (ksi)
3	8.87	6.49
7	11.28	7.96
10	13.18	9.44

of steel). It is shown that although HSRs have economic benefits, they increase the measure of displacements and drifts (the higher the grades of the steel, the further the drifts and displacements and the less the rigidity). To compensate this issue, it is necessary to increase the rigidity of members by increasing steel quantity or dimension of members which is a serious challenge and neutralizes the steel consumption reduction. It is also shown in pushover analysis that structures with less grade of steel, tolerates more shear than the ones with higher grades. Structures with higher grades of steel after further displacements enter the nonlinear stage. This means that the higher grade reinforcements tolerated more displacements. Finally, the strains of endpoints of beams which can be the representation of cracking phenomenon are acquired by nonlinear dynamic analysis with Opensees. It is observed that by increasing the grade of steel and number of the stories, the positive strains (tensile strains) which are proportionate to the crack width and crack spreading, increase.

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Please cite this article using:

A. Kheyroddin, H. Arshadi, F. Binaipur, An Overview of the Effects of High-Strength Reinforcement (HSR) on the Intermediate Moment-Resisting Frames, *AUT J. Civil Eng.*, 1(2) (2017) 177-188.

DOI: 10.22060/ceej.2017.12406.5207