# Assessing the Behavior of Concrete Moment Frames Reinforced with High-Strength Steel Rebar

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***Abstract :*** Materials used for structural members should achieve a quality approval from competent authorities under certain tests. Sometimes, in some projects, poor quality materials may be used due to poor supervision. When it comes to steel rebars, the lack of quality can be considered as higher resistance than nominal resistance, and the relative extension less than the minimum allowed. In this paper, the structural ductility has been evaluated in a way that structures have been designed based on standard material criteria, but in fact, inappropriate reinforcing steel rebars have been used in structure. In order to evaluate such circumstances, a typical material behavior is considered and numerical models analyzed in nonlinear static (with monotonic and cyclic loading) and dynamic (time-history) methods both for standard and non-standard rebars. It was observed that using the rebars with higher yield stress and the relative lengthening less than the standard value, despite the increase in the resistance of the overall structure, its ductility is reduced.

***Keywords*: Displacement Ductility, Reinforced Concrete Moment Frame, Steel Rebar, Nonlinear Analysis of Structure.**

## I. INTRODUCTION

Concrete is one of the most important construction materials in the modern world. Due to the defect in its tensile strength, this material is needed to be reinforced by embedding steel rebars. Steel itself, with a reputation even more than concrete, is an expensive construction material which is used in various range of structures from bridge piles to barrel vaults, [1-2] Initial geometric and material imperfection is a common defect and may cause unexpected structural behavior, buckling, or expedite fatigue crack initiation in steel structures. Thus, researchers continually study structures to ensure their performance and reliability. For instance, Arabi et al. have investigate the fatigue behavior of dynamic sign support structures during transportation;[3],[4]; and during the service life of these structure under various environmental loads;[5],[6],[7].

Over the past several decades, in many countries, the reinforced concrete structures have been erected by rebars with 414 Mpa (60 ksi) yield strength. The yield strength of 517 Mpa (75 ksi) has been rarely utilized, especially for the columns. Nevertheless, in a number of previous studies, the use of higher resistance rebars in beams, joists and columns have been investigated.

Richart F et al. [8] published their research results on reinforced concrete columns. In these columns, longitudinal rebars had 496 Mpa and 662 Mpa yield strength and specimens demonstrated a good performance under axial loads. They tested a series of reinforced concrete columns with circular cross sections under axial loads without eccentricity. They concluded that such compressive stresses up to 662 Mpa are induced in longitudinal rebars.

Todeschini et al. [9] published their research results on concrete columns with external loading. In their study, the columns were similar to those of Richart F et al. [8] experiment, but with rectangular cross sections, and the results illustrated that the samples had a good performance under the external loads and with a yield resistance of 620 Mpa.

Between the 1950s and 1960s, the Portland Cement Association (PCA) conducted a series of experiments.The goal of these experiments were to evaluate moment strength, moment cracking control, concrete crushing under compressive stress and creep in reinforced concrete samples. The rebars used in this study were in the range of 380 Mpa to 827 Mpa.

Simultaneously, Thomas [10] published their research results about reinforced concrete beams. The yield stress of the rebars used in this study was 1568 Mpa. The obtained results were applied in the 1971 edition of the building code requirement of ACI 318, [11]. In this version, the highest yield strength for steel bars was increased to 550 Mpa.

At that time, the properties of steel bars with the resistance greater than 517 Mpa had been explicitly mentioned in the American Society for Testing and Materials (ASTM) standard, [12]. However, the upper limit of yield stress of the steel bars in special moment frames was maintained at 413 Mpa.

In the initial tests in which steel bars were used in reinforced concrete members, the loading was monotonic, in which the load was applied incrementally and in one direction to the test sample. There was not any cyclic loading in these experiments. This type of loading was first carried out in the early 1960s, and by Burns & Siess. These experiments were conducted to study the performance of reinforced concrete members under seismic loadings. A huge number of experiments were carried out during the 1970s and 1980s.These studies illustrated that more ductile rebars are needed to deal with the forces resulted by earthquake.

In the early 1970s, the Committee of the Structural Engineers Association of California (SEAOC) recognized the need of applying the rebars with limited tensile properties. This limitation was applied to the final and yield resistance of steel bars to ensure the performance of the weak beam-strong column of moment frames.

Several experiments have been carried out on reinforced concrete columns under cycling loading, but almost all of the samples in the experiments have been reinforced with the bars having 413 Mpa yield stress. In 2006, Restrepo et al. [13] carried out a series of experiments, studying the behavior of columns with high strength reinforcements. In these experiments, two columns were examined: one with a yield stress of 413 and another with 827 Mpa. In both of these samples, the stirrups were identical to the longitudinal bars. In other words, both of them had the same yield stress. During the experiment, in the drift of 3.1 %, one of the stirrups was dispatched away, resulting in buckling of the longitudinal bar, and the column collapse in the drift 3.9%. The column in which the yield stress of the bars was 413 Mpa came to about 5% drift before the collapse occurred. The comparison of the results obtained from this test is rather difficult, as the failure in the columns with high resistance bars was due to the welding rupture occurred in the stirrup and it cannot be attributed to the mechanical properties of the bars.

## II. CHALLENGES OF USING HIGH-STRENGTH STEEL REBARS

One of the reasons that concrete building codes in 1963 limited the yield resistance of steel rebars to 413 Mpa was to control the width of the cracks under service loads. The experiments performed by Hognestad [14] presented that both the maximum and the average amount of the cracks' width are proportional to the amount of stress produced in steel rebars. According to those results, the American Concrete Institute (ACI) restricted steel rebars maximum yield stress to 413 Mpa to control the width of the cracks.

Another reason for restricting the maximum yield strength of the rebars is the reduction of bending stiffness of the reinforced concrete member due to using high-strength steel rebar. The bending stiffness of a reinforced concrete member is affected by the amount and position of the longitudinal rebars. When high-strength steel rebars are used, less steel is needed to reach the desired bending capacity. This reduces flexural hardness after cracking which affects the moment - curvature diagram of reinforced concrete section,[15].

## III. NUMERICAL MODELS

In this paper, to investigate the influence of rebars with higher yield stress and less ductility on overall behavior of the structures, four numerical models were considered. Models were designed for three and six story buildings and with considering the standard behavior of rebars with a yield stress of 400 Mpa. Figure 1 shows the assumed dimension for building. The span of moment frames in both X and Y directions are 6.4 meters and the story height is 3.2 m. The structural system is an intermediate moment frame and the gravity load caring system is two-way slab with 20 cm thickness. The models are designed for 21 Mpa concrete compressive strength. The assumed dead load of 650 Kg/m2 and live load of 200 Kg /m2 for each story. The load combinations are considered according to the ASCE07-16 instruction, (American society of civil engineers 2014), [16]. The models are designed according to ACI 318-08 code, [17].

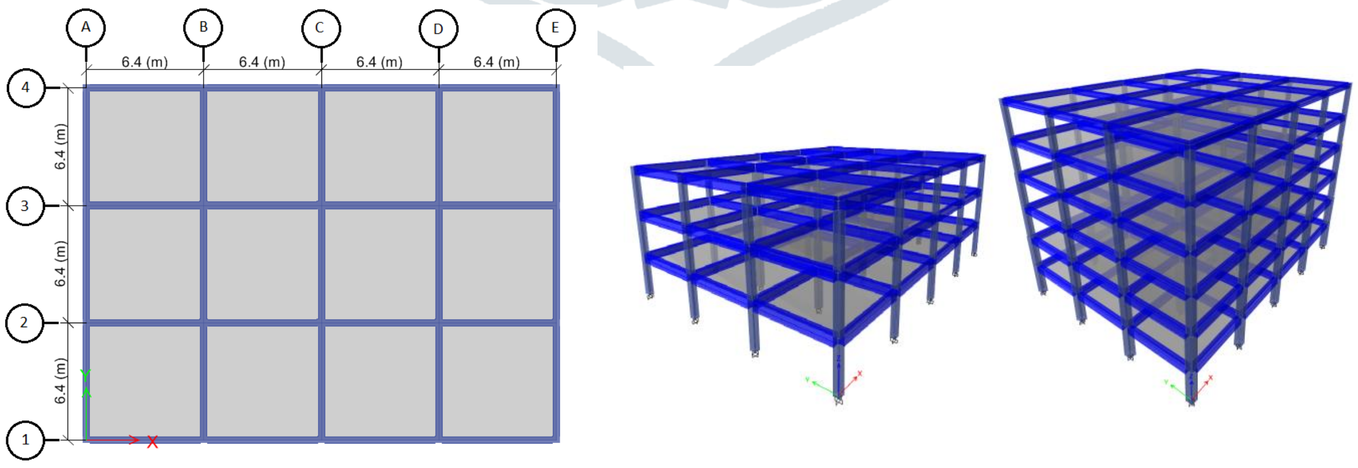
## IV. NONLINEAR ANALYSIS

To assess the effect of high-strength rebars in overall behavior of the structure, models are analyzed by considering the nonlinear behavior of materials. To simulate structural behavior of models Opensees (Open System for Earthquake Engineering Simulation) software, was utilized. The introduction of nonlinear materials in this software is by defining the stresses and strain values of the material in the both elastic and inelastic ranges. In the library of Opensees there are different commands to define various types of materials. In this study, concrete01 command were used to define concrete material,[18]. This command utilizes the suggested relation of Kent-Scott-Park to define the stress and strain values of concrete, in which the tensile resistance of concrete has been neglected.

Standard rebars for longitudinal reinforcements of beams and columns, are defined by ReinforcingSteel command and Hysteretic material command has been used to define the properties of steel materials. The yield strength is considered 580 Mpa exceeding the standard value of 413 Mpa and ultimate strain of 5.8% which is less than the allowed limit of 14%. Figure 2 compares the behavior of defined material both for standard and high-strength steel rebars for monotonic and cyclic loading condition.

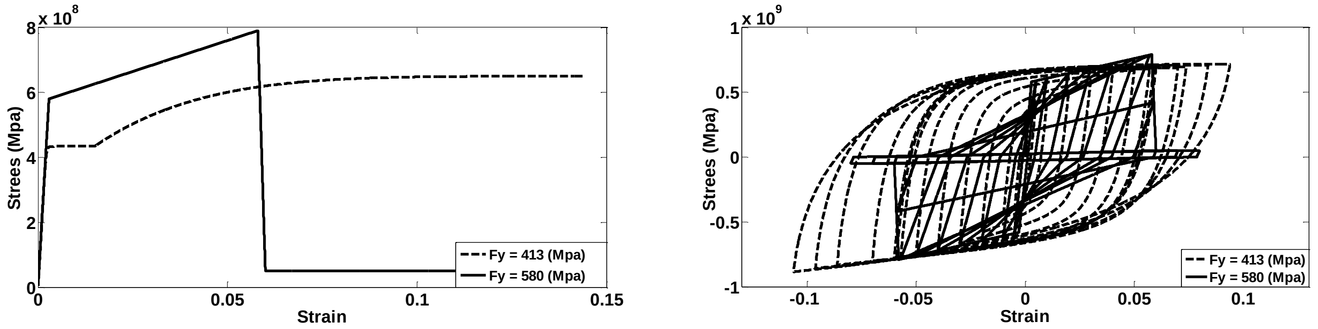
### 4.1 Nonlinear Static Analysis with Monotonic Lateral Load (Pushover)

Pushover analysis is an approximate analysis that gives an estimation of the lateral strength of the structure. The analysis starts with applying gravity loads to the model and holding these loads constant until the end of the analysis. In the next step, lateral loads are applied to the model which is representative of the loads created by earthquake in each story level of the structure.




(a) (b)

**Figure. 1 Numerical models : (a) plan view (b) 3D view of 3-story structure and 6-story structure**




(a) (b)

**Figure. 2 Behavior of defined material for standard and high-strength steel rebars under (a) monotonic and (b) cyclic loading condition.**

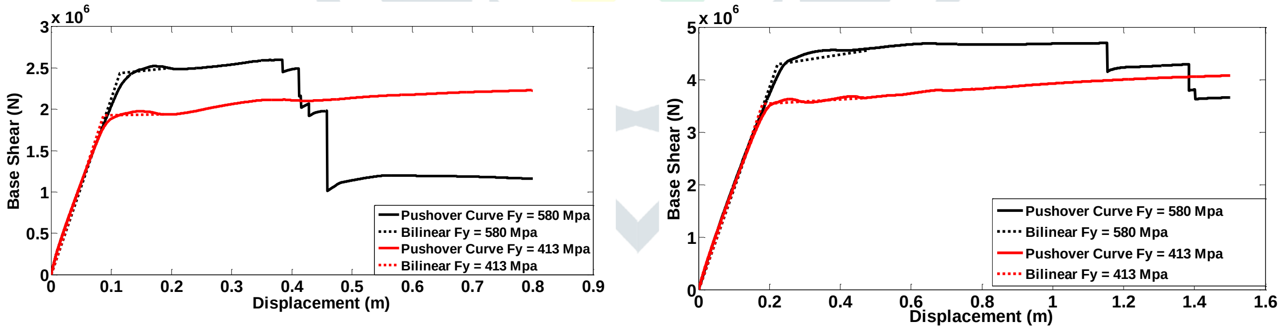
The lateral loads are increased gradually until the roof displacement reaches a defined value (target displacement) or the whole structure collapse. In this study, invers triangular load distribution was considered as lateral forces and the models are pushed more than target displacement to compare the overall behavior of the structures in their ultimate capacity region. The result of analysis is capacity (or pushover) curve which is illustrated in figure 3 for all numerical models. In this figure the exact curve is idealized to bilinear curve by obtaining the values of Δy, Vy, Δu and Vu according to Prestandard and Commentary for the Seismic Rehabilitation of Buildings, FEMA-356, [19]. These values are provided in table 1 for both 3-story and 6-story models. The displacement ductility of structure for each model is achieved through dividing the final displacement (Δu or target displacement) over the yield displacement (Δy).

### 4.2 Nonlinear Static Analysis with Cyclic Lateral Load

This analysis, similar to the pushover, is a static analysis with a difference that the direction of lateral loads changes when the displacement reaches to determined values. The reversed cyclic loads were applied under displacement control. In this research the loading protocol which is considered for applying displacement at top of the models includes 7 cycles. First three cycles are in the elastic phase while the other 4 cycles are in the plastic phase of the pushover curve. Figure 4 plots the applied lateral drift routines for both 3-story and 6-story models in each cycle. Figure 5 shows cyclic behavior of models and figure 6 compares the energy dissipation of models during each cycle.

**Table 1. Results from pushover analysis**

| | 3-story model with Fy = 413 Mpa rebars | 3-story model with Fy = 580 Mpa rebars | 6-story model with Fy = 413 Mpa rebars | 6-story model with Fy = 580 Mpa rebars |
|---|---|---|---|---|
| $\Delta_y$ (m) | 0.0875 | 0.1142 | 0.1800 | 0.2235 |
| $V_y$ (KN) | 1928.7 | 2437.3 | 3536.8 | 4291.8 |
| $\Delta_u$ (m) | 0.1920 | 0.1980 | 0.4674 | 0.4775 |
| $V_u$ (KN) | 1937.0 | 2496.0 | 3652.4 | 4553.6 |
| μ | 2.1947 | 1.7341 | 2.5965 | 2.1363 |




(a) (b)

**Figure. 3. Pushover and idealized bilinear curve for (a) 3-story and (b) 6-srory models.**

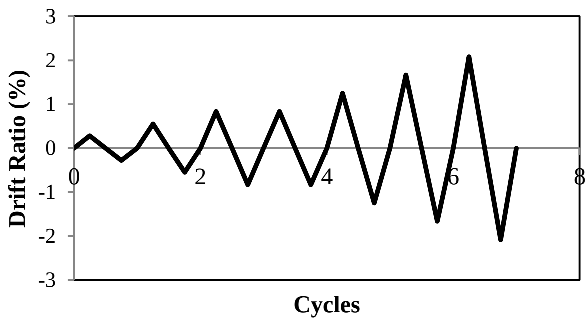



**Figure. 4 Lateral drift routines for 3-story and 6-srory models.**

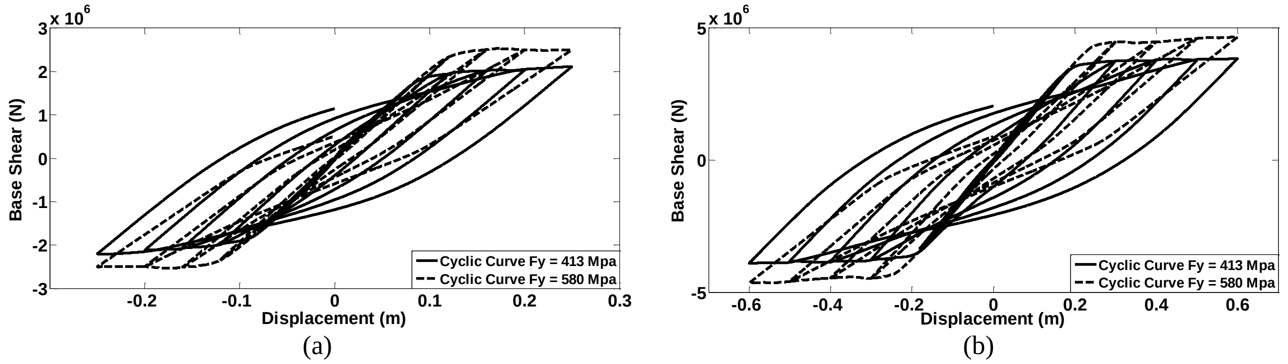



**Figure. 5 Cyclic behavior of (a) 3-story and (b) 6-srory models**

### 4.3 Nonlinear dynamical analysis

In the next step, behavior of the models is assessed with nonlinear time-history dynamic analysis. For this purpose, seven ground motions are selected from Pacific Earthquake Engineering Research Center (PEER) database which the name of the selected motions is provided in table (2). Earthquakes are selected in a way that their intensity is between 5.8 to 6.8 Richter scale with a distance of 25 to 75 kilometers from the epicenter. The soil type is assumed to be dense to medium.

After normalizing the accelerograms to 1.0g and obtaining the scaling factor, accelerograms are modified and nonlinear time-history analysis performed for each model. Considering the great number of the results, figure 7 shows the roof displacement of the 3-stroy and 6-story models for the Chi Chi earthquake as an instance. In order to evaluate the rate of damage in each earthquake, number of plastic hinges which is formed during the earthquake are recorded and provided in table 3for 3-story and 6-story models.

**Table 2. Selected ground motions for time-history analysis**

| Earthquake Name | Year | Station Name | Magnitude | Mechanism | Rjb (km) | Rrup (km) | Vs30 (m/sec) | 5-95% Duration (sec) |
|---|---|---|---|---|---|---|---|---|
| *Big Bear-01* | 1992 | Desert Hot Springs | 6.46 | strike slip | 39.52 | 40.54 | 359.00 | 12.1 |
| *Chi-Chi_ Taiwan-04* | 1999 | CHY015 | 6.20 | strike slip | 50.02 | 50.04 | 228.66 | 25.6 |
| *Imperial Valley-06* | 1979 | Coachella Canal #4 | 6.53 | strike slip | 49.10 | 50.10 | 336.49 | 11.1 |
| *Livermore-01* | 1980 | Fremont - Mission San Jose | 5.80 | strike slip | 34.66 | 35.68 | 367.57 | 10.1 |
| *Morgan Hill* | 1984 | Capitola | 6.19 | strike slip | 39.08 | 39.08 | 288.62 | 15.3 |
| *Northern Calif-01* | 1941 | Ferndale City Hall | 6.40 | strike slip | 44.52 | 44.68 | 219.31 | 15.5 |
| *Superstition Hills-02* | 1987 | Calipatria Fire Station | 6.54 | strike slip | 27.00 | 27.00 | 205.78 | 12.6 |

**Table 3. Number of plastic hinge occurred during the earthquake**

| | 3-story | | 6-story | |
|---|---|---|---|---|
| | Rebar Fy = 413 Mpa | Rebar Fy = 580 Mpa | Rebar Fy = 413 Mpa | Rebar Fy = 580 Mpa |
| *Big Bear* | 17 | 19 | 39 | 41 |
| *Chi Chi* | 21 | 22 | 44 | 47 |
| *Imperial Valley* | 18 | 17 | 9 | 4 |
| *Livermore* | 6 | 4 | 31 | 37 |
| *Morgan Hill* | 4 | 2 | 6 | 1 |
| *Northern Calefornia* | 5 | 4 | 6 | 2 |
| *Superstition Hills* | 0 | 0 | 0 | 0 |

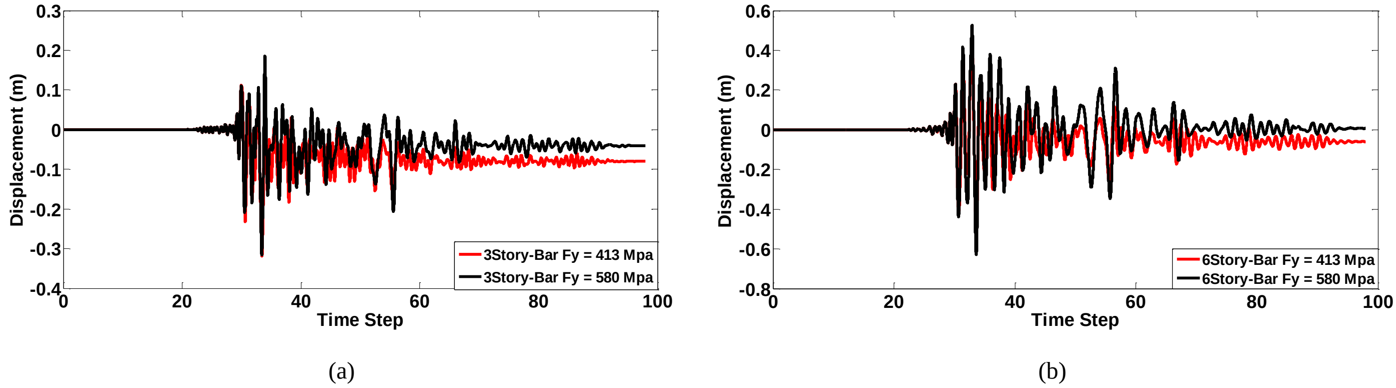



**Figure.7 Models behavior under Chi-Chi earthquake for (a) 3-story model and (b) 6-story model**

## V. CONCOLUSION

This research was to analyze the effects of steel rebars on the behavior of reinforced concrete moment frame through a variety of static and dynamic nonlinear analyzes. Considering a simple architecture plan, a reinforced concrete structure with the intermediate moment frame was designed for two types of 3 and 6-story buildings. By selecting the moment frames along the longitudinal axis as representation of the entire structure, the effects of the longitudinal rebars behavior on structural ductility in a two-dimensional space was investigated. Considering rebar with 413 Mpa yield stress as standard rebar, the second rebar was chosen in a way that its yield stress exceeded the standard (580 Mpa) and its minimum extension at ultimate load is less than the allowed value determined by the standard criteria. The models were analyzed through nonlinear static (with monotonic and cyclic loading) and dynamic analysis. The following outcomes are achieved from this study:

- The final resistance in the structures with high-resistance rebars is 28% and 25% greater than the models with standard rebars for 3 and 6-story models respectively.
- The ductility of the structures with high-resistance rebars decreases by 21% and 18% for 3-storey and 6-story models, respectively.
- The use of high-resistance rebars with a less extension length from minimum allowed makes the energy dissipated in the structure decrease. This reduction is close to 80% for 3-story and 63% for the 6-story models.
- In 3-story models, the drift in floors (except for the third floor) for the models with high-resistance rebars is a little (almost 5%) lower than those with standard bars.
- In 6-story models, the drift for the models with high-resistant rebars in all levels are 22% more than models with standard rebars.
- By comparing the states 4 and 5, it seems to be concluded that in the lower structures, since the final strength is more effective than the structural ductility, the effect of using the high-resistance rebars is positive and the structure behaves more appropriately. However, after increasing the structure’s height, its ductility plays a more significant role in the oveall behavior of the structure, and since in the structure with the high-resistance bars ductility is less than that of structures with standard rebars, this issue plays a negative role in the behavior of the structure.
- Damage occurred in devastating earthquakes in structures with high-resistance rebars is more than those with standard rebars. This is understandable by comparing the number of plastic hinges formed in 3-story and 6-story structures influenced by such earthquakes as Big Bear and Chi Chi.
- Using reinforcing rebars beyond the standard limit and with less extension length than the minimum allowed value can lead to the negative performance of the structure. This is more tangible in high-altitude structures under severe earthquakes. Hence, if a structure uses such rebars, it is necessary to utilize rehabilitation approaches to increase the structural ductility.

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