# Numerical Investigation of Deflection Amplification Factor in Moment Resisting Frames Using Nonlinear Pushover Analysis

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**Abstract** – *The lateral deflections were obtained from an elastic analysis under seismic loads are much lower than actual deflections which building structures must withstand during strong ground motions. Recent building codes increased the elastic deflection by an amplification factor which is a parameter of over strength factor and ductility ratio. The over strength factor is a coefficient which applied to design forces to find the maximum base shear after forming plastic hinges, and ductility ratio is the capability of structures to withstand nonlinear deflection before the collapse. These seismic parameters can be obtained performing nonlinear pushover analysis and using capacity curve. In this research, five ordinary and special moment resisting frames with three, five, seven, nine, and twelve stories have been subjected to a nonlinear pushover analysis and their capacity curves has been derived. The calculated values for the over strength factor and deflection modification factor were compared with the introduced values in ASCE 7-10. The results of this comparison indicate that ASCE 7-10 values are slightly conservative.*

**Keywords-** *Deflection Amplification Factor, Over strength Factor, Ductility Ratio, Nonlinear Pushover Analysis, Ordinary Moment Resistant Frame, Special Moment Resistant Frame.*

## INTRODUCTION

**T**he seismic design of steel structures was radically changed after the Northridge 1994 earthquake in California, United States. After this earthquake, based on the observed damages in the existing steel structures, special seismic design specifications were established in the American Institute of Steel Construction, AISC, and Uniform Building Code, UBC [1,2].

Overstrength factor, $\Omega_0$, is an important coefficient in estimating the maximum lateral displacement for the building and bridge structures under seismic loads [3]. Considering overstrength factor, ductility ratio and deflection amplification factor in the design of new components in building and bridge structures lead to less damage under vibration loads during the service life [4-7]. Nonlinear static and incremental nonlinear dynamic analyses are common numerical methods for an estimation of seismic parameters [8]. Soltangharaei et al, estimates behavior factors for special moment resisting frames, for near and far fault ground motions using

incremental dynamic analysis [9-10]. Although incremental dynamic analysis resembles more realistic seismic behavior of structures compared to pushover analysis, it is much more complex and time costly than pushover analysis, and highly sensitive to ground motion record selection. In this research, several steel moment resisting frames subjected to nonlinear pushover analysis. The effective parameters that control lateral displacement of structures under seismic loads are described using the capacity curve. Finally, the calculated values from the capacity curves were compared with the prescribed values in the ASCE 7-10.

## 2. SEISMIC DESIGN CONCEPTS USING CAPACITY CURVE

Current seismic design practice is based on analysis results from the elastic analysis which decreased by implementing response modification factor [11]. In this method, some of the components of the structure are designed in such a way that acts as a fuse against seismic loads, which means that these members are placed in inelastic-plastic range and energy dissipated through the plastic deformation in predefined members. These

calculated based on the codes. Due to the dissipated energy by nonlinearity in some internal elements, the design force reduced to $C_s$ for LRFD method and $C_w$ for ASD method. This reduction to the level of design forces takes place by applying response modification factor $R_u$ for LRFD method and $R_w$ for ASD method. For example, in the LRFD method, design force is obtained by dividing the maximum elastic base shear, $C_{eu}$, by $R_u$.

$$C_s = \frac{C_{eu}}{R_u} \qquad\qquad\qquad\qquad\qquad\qquad (1)$$

The first plastic hinge forms in the structure at the design force, $C_s$. By increasing the lateral load beyond design force, $C_s$, other members in the structure potentially enter in the nonlinear range, and the whole structure response curve would show nonlinearity behavior. The capacity curve of the structure idealized by a bilinear curve which represents the elastic-plastic behavior. The bilinear curve is represented by the dotted lines in Figure 1. This bilinear curve shows that $R_u$ dependents on other coefficients as below.
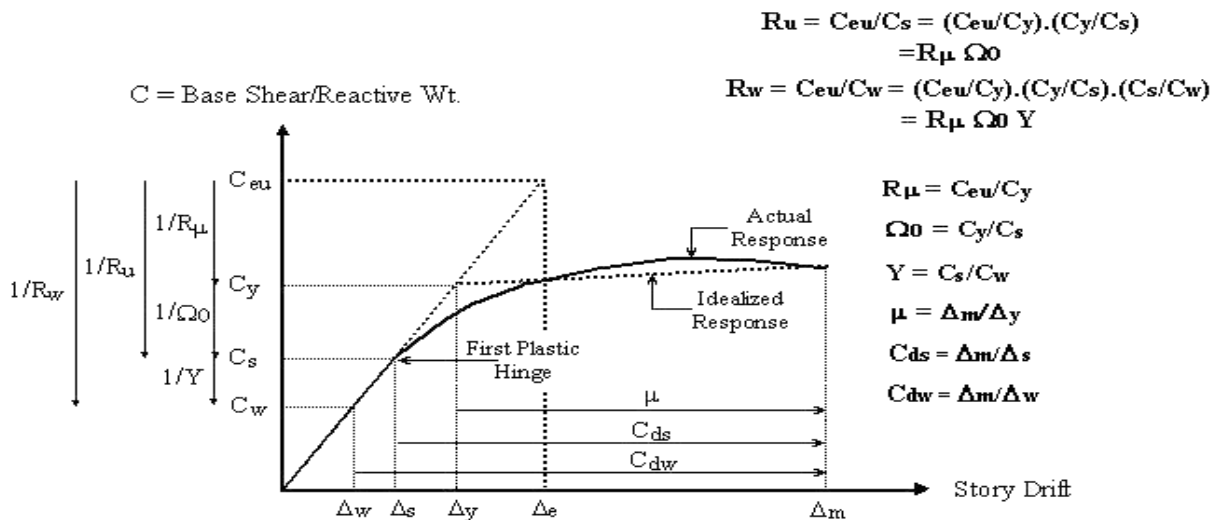


Figure 1. Capacity curve and different force levels.

members are weak links of the system and entered in inelastic-plastic range, while the other members and their connections must remain inelastic range.

Figure 1 shows the capacity curve of a structure. In this curve, the horizontal and vertical axis represents the story drift and the base shear, respectively. If it was assumed that the structure is designed to behave inelastic during seismic loads, the maximum elastic base shear, $C_{eu}$, would be much larger than the design force level

$$R_u = R_\mu \Omega_0 \qquad\qquad\qquad\qquad (2)$$

$$\Omega_0 = \frac{\Delta_y}{\Delta_s} \qquad\qquad\qquad\qquad (3)$$

$$\mu = \frac{\Delta_m}{\Delta_y} \qquad\qquad\qquad\qquad (4)$$

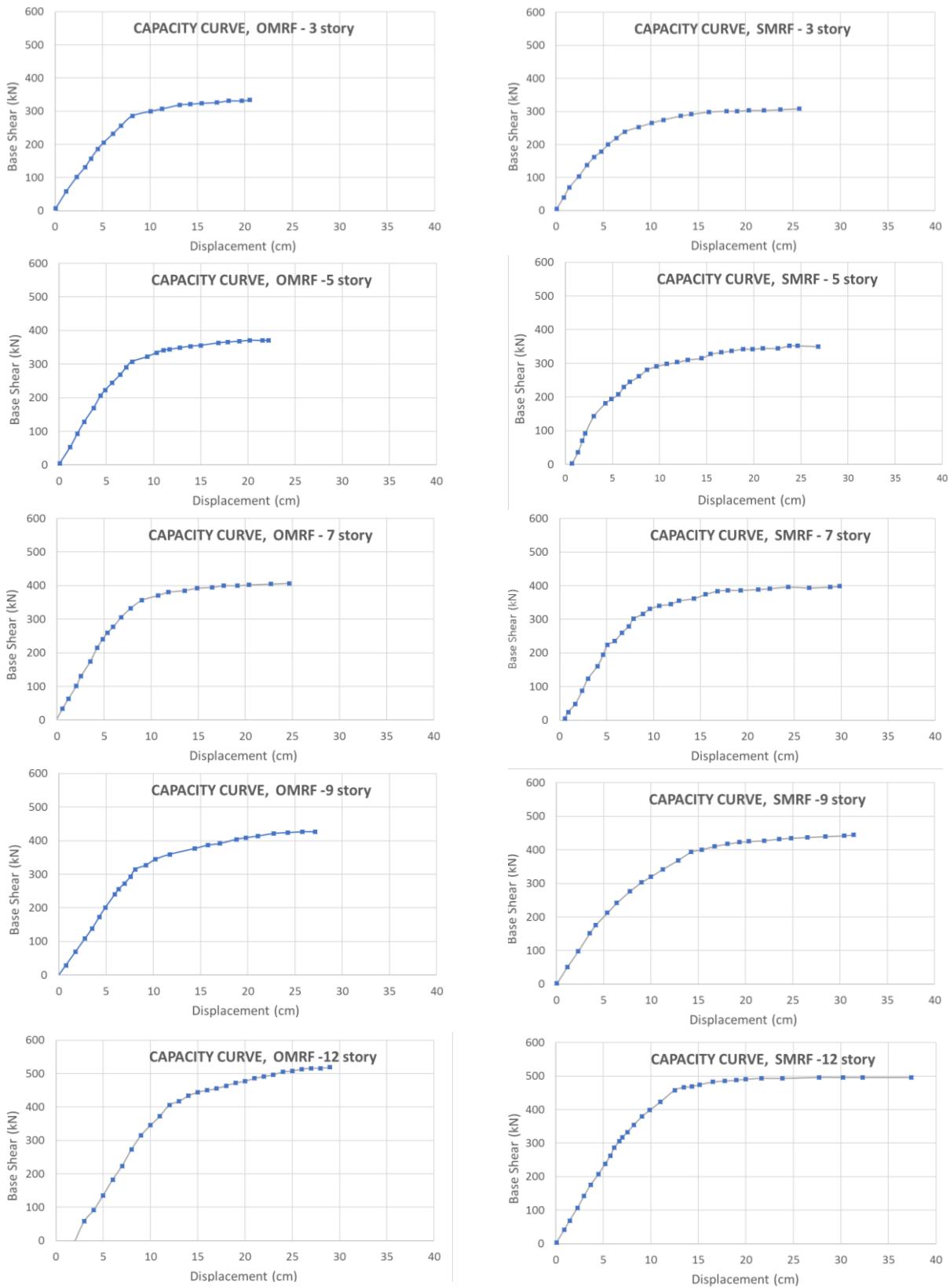Ductility reduction factor, $R_\mu$, reduces seismic forces

Figure 2. The capacity curve of the Ordinary (OMRF) and Special (SMRF) Moment Resisting Frames.

From $C_{eu}$ to $C_y$ level in the capacity curve. This capability is provided by elements of a structural system designed to dissipate energy [12]. Miranda and Bertero have proposed equations to estimate $R_\mu$ at a 5% damping

on different soil types [13]. Also, Krawinkler and Nassar developed equations for obtaining this coefficient on a rock or stiff soil at 5% damping [14]. In this research, this coefficient is calculated using the proposed equations by New Mark and Hall[15]. These equations are as below:

$$R_\mu(T \le .03) = 1.0 \quad\text{.............................(5)}$$

$$R_\mu(.12 \le T \le .5) = \sqrt{2\mu - 1} \quad\text{................(6)}$$

$$R_\mu(1 \le T) = \mu \quad\text{...............................(7)}$$

## 3. ESTIMATING MAXIMUM LATERAL DISPLACEMENT

The philosophy behind seismic loads in the most design codes rely on the ductility of structures in the nonlinear range of the capacity curve. In other words, they reduce the value of design forces using the inherent capability of structure to dissipate input energy.

For steel and reinforced concrete buildings material ductility, cross-section ductility, member ductility, and structure ductility are widely used [16,17]. While in timber constructions since wood is an inherently brittle material and timber elements exhibit almost no potential for energy dissipation. Thus, in a timber structure the only elements that provide ductility, and consequently exhibit hysteretic dissipation of energy under cyclic loading, are the metal connection systems as documented by many researchers [18-20]. This philosophy is applied to other non building type structures facing extreme loading condition but are beyond the scope of this work[21,22].

The response modification factor, $R$, is considered in seismic design to reduce maximum elastic base shear, $C_{eu}$, due to the nonlinear behavior of the structure. Since the structure enters in the nonlinear range during seismic loads, the actual displacement would be greater than the values calculated based on an elastic analysis. For this purpose, to find the maximum lateral displacement, the deflection amplification factor introduced to increase displacement calculated from an elastic analysis. The relationship between this coefficient and the response modification factor in Load and Resistance Factor Design (LRFD) is as below:

$$C_{ds} = \frac{\Delta_m}{\Delta_s} = \frac{\Delta_m}{\Delta_y} \times \frac{\Delta_y}{\Delta_s} = \mu\Omega_0 \quad\text{.................(8)}$$

$$\Delta_m = C_{ds}\Delta_s = \mu\Omega_0\Delta_s \quad\text{.....................(9)}$$

Now, by calculating the ductility coefficient, $\mu$, and$\Omega_0$, the deflection amplification factor, $C_{ds}$, can be determined. By using the New Mark Hall equation for$R_\mu$ and for the structure with long periods, $R_\mu = \mu$ then:

$$\Delta_m = R_\mu\Omega_0\Delta_s \quad\text{.....................(10)}$$

It means the real displacement is equal to:

$$\Delta_m = R_u\Delta_s \quad\text{.....................(11)}$$

Based on the code provisions, the nonlinear lateral displacement can be estimated by applying deflection amplification factor,$C_d$, to the displacements calculated from the elastic analysis of the structure under a determined design force. In the seismic design codes, the expected maximum nonlinear displacement is equal to:

$$\Delta_m = \beta R_u\Delta_s \quad\text{.......................(12)}$$

Which $\beta$ is an experimental coefficient.

## 4. CODE SPECIFICATION FOR DEFLECTION AMPLIFICATION FACTOR, $C_d$

### 4.1 UBC97 Code

In the UBC97 standard, the nonlinear displacement value is determined as below [2]:

$$\Delta_m = 0.7R_u\Delta_s \quad\text{.......................(13)}$$

This equation is similar to the Equation 12, in which $\beta = 0.7$ and it can be rewritten as:

$$C_d = 0.7R_u \quad\text{.......................(14)}$$

### 4.2 ASCE 7-10

However, theoretically, using Equation 13 is logical to determine the nonlinear displacement of structures, but it is better to consider the effect of other parameters, such as the importance of the structure and the earthquake resisting system, in determining this value. ASCE 7-10 has followed such an approach [23]. In this case, the nonlinear displacement coefficient is determined from the following equation.

$$\Delta_m = \frac{C_d\Delta_s}{I} \quad\text{.......................(15)}$$

which $I$ is the importance factor of the structure and $C_d$ is the deflection amplification factor corresponding to the earthquake resisting system.

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## 5. NONLINEAR PUSHOVER ANALYSIS OF CASE STUDY FRAMES

For extracting the deflection amplification factor, first, the overstrength factor and the ductility ratio of the structure should be determined. For this purpose and comparing the results with values provided by the Code, five different moment resisting frames (MRF) with different stories such as 3, 5, 7, 9 and 12 were modeled and designed using CSI SAP2000 program [24]. Each of these structures was analyzed and designed in accordance with the ordinary moment resisting frame and special moment resisting framespecifications [25], then a nonlinear pushover analysis carried out, and the lateral load pattern corresponding to ASCE 7-10 vertical distribution of seismic forceswas increased step by step until the structure collapsed or nonlinear analysis did not get converge. Nonlinear pushover analysis was performed using advanced nonlinear features and analysis of CSI SAP2000 program [24]. The nonlinear pushover analysis can capture the failure modes, defining the location of plastic hinges and plotting the capacity curve. The results of the nonlinear pushover analysis are shown in Figure 2 for OMRF and SMRF. The overstrength factor and ductility ratio were extracted from the above diagrams for each of the ordinary and special moment resisting frames according to Table 1.

Table 1.Overstrength factor and ductility ratio of ordinary and special MRF.

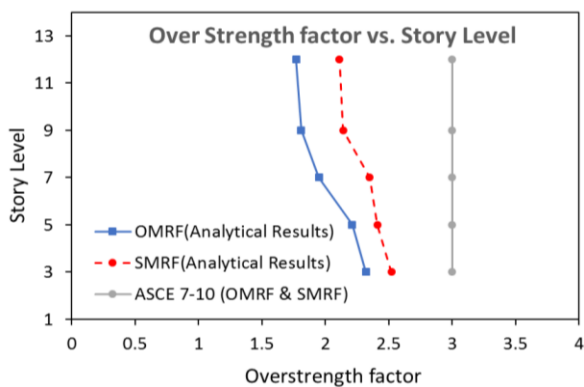| No. of Stories | Overstrength Factor | | Ductility ratio | |
|---|---|---|---|---|
| | OMRF | SMRF | OMRF | SMRF |
| 3 | 2.32 | 2.52 | 1.05 | 1.99 |
| 5 | 2.21 | 2.41 | 1.29 | 2.08 |
| 7 | 1.95 | 2.35 | 1.48 | 2.22 |
| 9 | 1.81 | 2.14 | 1.68 | 2.57 |
| 12 | 1.77 | 2.11 | 1.79 | 2.64 |



Figure 3. Overstrength factor for different story levels for ordinary and special MRF.

Figure 3 shows the overstrength factor versus story level that indicates ductility decreased by number of story levels or by increasing the period of structure. Figure 4 shows the ductility ratio calculated based on capacity curves. Figure 5 shows the deflection amplification factor for the ordinary and special moment resisting frames resulted from analytical investigation and suggested value by ASCE 7-10.
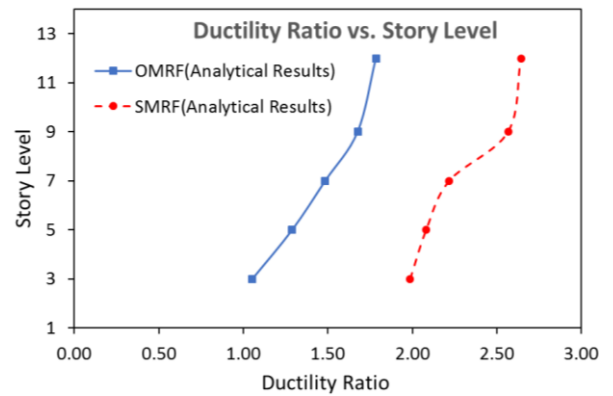


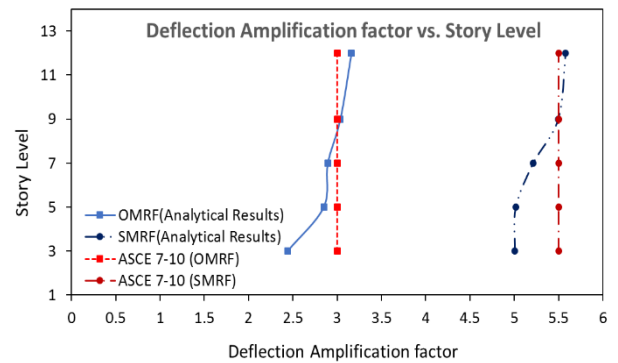Figure 4. Ductility ratio for different story levels for ordinary and special MRF.



Figure 5. Deflection amplification factor for different story levels in ordinary and special MRF.

## VI- CONCLUSION

In this research, five-moment resisting frame with three, five, seven, nine, and twelve stories have been subjected to a nonlinear static analysis and their capacity curves has been determined. Then, the overstrength factor and ductility ratio derived using the capacity curves. Finally, the deflection amplification factor was obtained. According to the analysis results, it was observed that the over strength factor decreases with increasing number of stories. It approximately remains constant for frames with nine to twelve story levels.

The SMRF has higher ductility ratio and overstrength factor than OMRF. For both OMRF and SMRF the overstrength factor in the ASCE 7-10 is higher than the

obtained overstrength factor from the analytical results, therefore ASCE 7-10 is slightly conservative. The SMRF have higher $C_d$ compared to OMRF according to analytical results, however, values suggested by code is higher.

The deflection amplification factor has a direct relationship with the structural ductility ratio. In this study, the derived deflection amplification factors are less than prescribed values in ASCE 7-10 for frames below 8-story, and with increasing the number of stories, this coefficient becomes closer to the values introduced in the Code. This value almost corresponds to the values introduced in the Code for the SMRF and OMRF in the eight-story frame.

The concept used in calculating the deflection amplification factor is consistent and equal, in both UBC97 and ASCE 7-10. In ASCE 7-10, the importance factor of the building is also included in the calculation of deflection amplification factor, which seems to have more logical meaning for calculating of this coefficient.

## NOTATIONS

The following notations are used in this study (Figure 1):

| | | |
|---|---|---|
| $C_{eu}$ | = | Maximum elastic base shear |
| $C_s$ | = | Design base shear coefficient in LRFD |
| $C_w$ | = | Design base shear coefficient in ASD |
| $C_y$ | = | Base shear at yielding point in idealized bilinear response |
| $R_u$ | = | Response Modification Factor in LRFD |
| $R_w$ | = | Response Modification Factor in ASD |
| $R_\mu$ | = | Ductility Reduction Factor |
| $\Omega_0$ | = | Overstrength factor |
| $Y$ | = | $C_s / C_w$ |
| $\mu$ | = | Ductility ratio, $\Delta_m / \Delta_y$ |
| $\Delta_s$ | = | Drift corresponding to design base shear coefficient in LRFD |
| $\Delta_w$ | = | Drift corresponding to design base shear coefficient in ASD |
| $\Delta_y$ | = | Drift corresponding to base shear at yielding point in idealized bilinear response |
| $\Delta_e$ | = | Drift corresponding to maximum elastic base shear |
| $\Delta_m$ | = | maximum Drift in the plastic range |
| $C_{ds}$ | = | $\Delta_m / \Delta_s$ |
| $C_{dw}$ | = | $\Delta_m / \Delta_w$ |
| $I$ | = | Importance factor |

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