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# DETERIORATION EFFECT OF HYSTERESIS LOOPS IN NONLINEAR STATIC ANALYSIS OF INTERMEDIATE AND SPECIAL STEEL MOMENT FRAMES

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## ABSTRACT

FEMA440 proposed coefficient method considers the effect of stiffness degradation and strength deterioration by $C_2$ modification factor. This factor is obtained by SDOF oscillator responses with few degrading hysteresis behaviors. This paper studies the ability of coefficient method to estimate displacement demands of MDOF buildings with mild and severe degradations. Performance of Intermediate and Special steel moment frames is compared separately by utilization of nonlinear static and dynamic analyses. Error values show a good correlation between nonlinear static and dynamic responses.

**Keywords**: Degradation; coefficient method; FEMA 440; welded connection

## 1. INTRODUCTION

Over the years, performance of steel moment fames with pre-Northridge connections was qualified for seismic design in engineering practice [1]. Nonetheless, the 1994 Northridge and a year later the Kobe earthquakes occurred. A building damage survey in the months after the earthquake, reported extensive damages mainly due to excessive lateral deformations, yielding, buckling and brittle fracture of beam-column connections [2]. Comprehensive studies on behavior of structural members are established since Northridge earthquake.

Foutch and Yun [3] used nonlinear dynamic and pushover analysis for two groups of structures. The former included nonlinear springs for connections and panel zones and the latter, modeled brittle behavior of pre-Northridge connection. Lee and Foutch [4] focused on performance prediction and evaluation of steel moment frames which were built prior to the Northridge earthquake. Ibarra et al. [5] described several degrading hysteresis models to consider degradation for calculation of inelastic responses of SDOF systems.

Nowadays, static nonlinear analysis is an effective tool for seismic evaluation purpose,

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since it provides adequate information on structure behavior. A well-known nonlinear static procedure is based on FEMA356 [6] and ASCE41-06 [7]. FEMA356 recommends the Coefficient Method, whereby displacement demands are obtained by modification of calculated elastic displacement demands. Later, FEMA440 [8] was commissioned to assess and modify nonlinear static procedure led to Modified Coefficient Method (MCM).

In this method, $C_2$ factor takes into account the amount of strength deterioration and stiffness degradation in hysteresis cycles. $C_2$ coefficient values have obtained by some basic hysteresis degrading behaviors. Since the characteristics of the hysteretic behavior are very sensitive to the structural material, detailing, and ground motion characteristics, determination of hysteretic behavior has an important role to evaluate demand displacements [8].

This study, attempts to evaluate the effects of various hysteretic degradations in steel moment frames to estimate displacement demands. Twelve intermediate and special steel moment resisting frames designed in accordance with Iranian building codes. Nonlinear dynamic analysis and nonlinear static analysis were carried out for comparison. Also $C_2$ modification factor was evaluated for these structures and compared with FEMA440 proposed values.

## 2. HYSTERETIC BEHAVIOR

In steel moment frames, degradation primarily comes from: (1) buckling; and (2) non-ductile connection behavior. Steel members with compact section and adequate lateral bracing show a bilinear hysteresis loop that consists of two linear elastic and nonlinear post-elastic states as is shown in Figure 1. This member shows stable hysteresis loops and is able to dissipate considerable amount of energy [9].

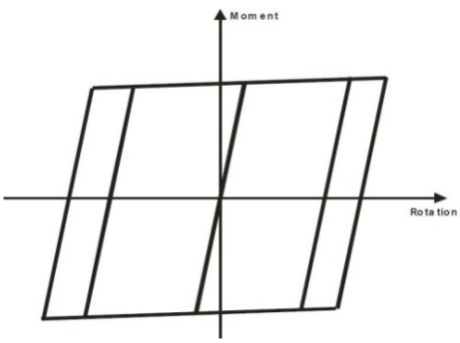


Figure 1. Bilinear elastic-plastic hysteretic model [9]

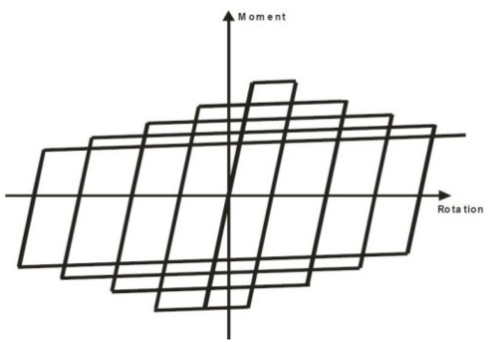


Figure 2. Strength degradation for bilinear hysteresis model [9]

Local buckling causes gradual degradation in strength. But the post-yield stiffness is not changed [10]. 10% to 40% decrease in strength is usually assumed for analytical modeling due to lack of experimental data [9]. Figure 2 shows a typical hysteresis rule of a steel member with local buckling. Also, another type of degradation is fracture of the weld in the beam flange which is observed extremely in pre-Northridge connections. Fracture is initiated at the beam flange weld, and may propagate to the column flange or web. Moment strength of the connection drops to a small portion of the plastic moment capacity due to fracture [11]. Reduction in moment capacity occurs when the crack is opened. By change in load sign, the initial strength is reversed [12]. Based on tests on steel connections, a few hysteresis models are still developed to capture post fracture behavior of steel moment connections. Figures 3 and 4 show two well-known typical hysteresis rules developed to model connection fracture which are developed by Kunnath [13] and Foutch and Shi [14], respectively.

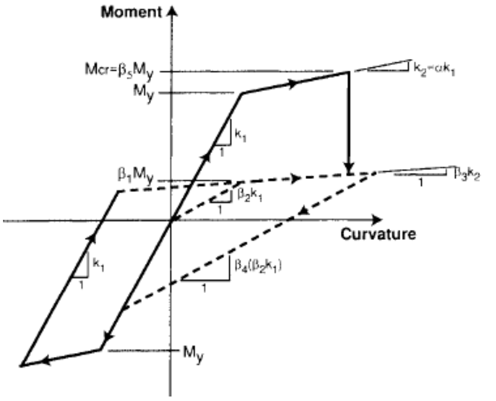



Figure 3. Hysteresis model for damaged welded connections [13]

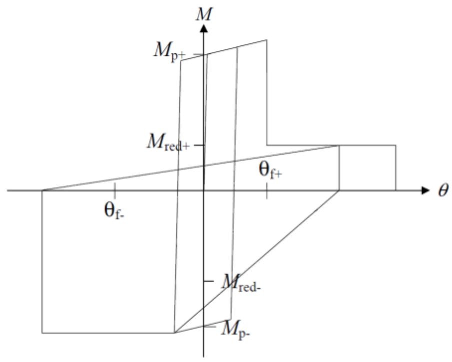
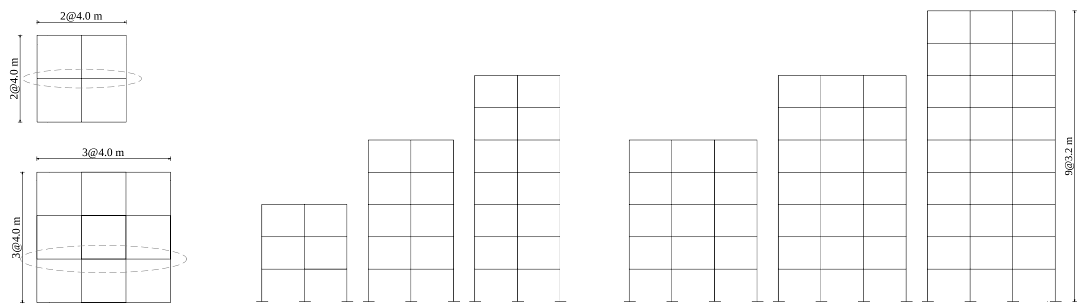



Figure 4. Hysteresis model for weld fracture [14]

## 3. DESCRIPTION OF SAMPLES

Two groups of intermediate and special steel moment frames were used in this study including 2-bay 3, 5, 7 and 3-bay 5, 7, 9 story frames. These frames are designed based on Iranian Code of Practice for Seismic Resistant Design of Buildings [15] and Iranian National Building Code, Part 10, steel structure design [16]. Models were considered to have stories with 3.2 m high and bays with 4.0 m length, located in high seismic risk areas with A=0.35g and soil type III [15]. Figure 5 plots the structures in plan and in elevation. It is supposed that steel's yield stress is 2400 *kg/cm²*. Floor's dead and live loads are 1100 *kg/m²* and 200 *kg/m²*, respectively.




(a) Plan view of frames

(b) Frames in elevation

Figure 5. Selected frames for analysis

## 4. INPUT RECORDS

A set of seven ground motion records are selected for nonlinear dynamic analysis with the minimum site distance in the range of 20-50 Km. All recorded on firm soil and include no directivity effects. Table 1 lists the ground motion records that are used for nonlinear dynamic analysis. The ground motions are scaled to fit the design spectrum in the range of 0.2T to 1.5T according to Iranian code of practice [15]. Figure 6 shows the scaled ground motions.

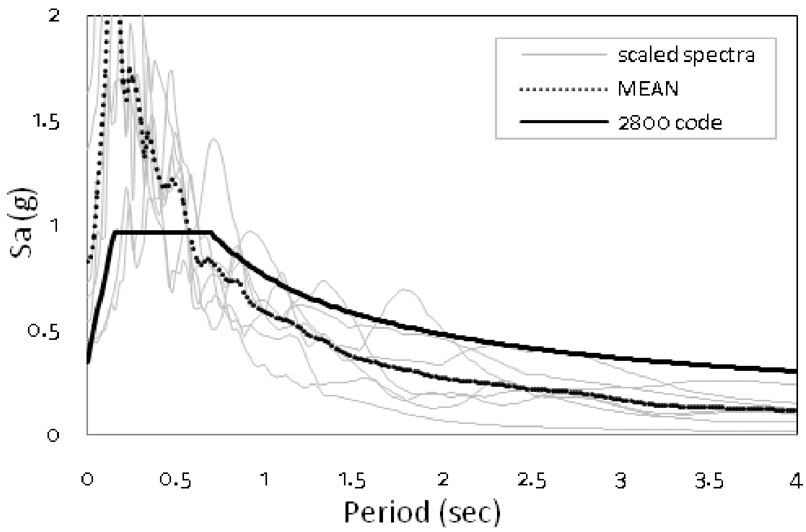



Figure 6. Scaled ground motion spectra.

Table 1: Specifications of used ground motion records.

| No | Year | Earthquake | Magnitude | Station | Dist. (km) | PGA (g) |
|---|---|---|---|---|---|---|
| 1 | 1971 | San Fernando | 6.61 | Hollywood Stor FF | 39.49 | 0.44 |
| 2 | 1979 | Imperial Valley | 6.53 | EL Centro #5 | 27.80 | 0.26 |
| 3 | 1983 | Coalinga | 6.36 | Cantua Creek School | 30.06 | 0.26 |
| 4 | 1987 | Whittier Narrows | 5.99 | Union Oil | 24.32 | 0.36 |
| 5 | 1989 | Loma Prieta | 6.93 | Hollister Diff. Array | 45.10 | 0.28 |
| 6 | 1994 | Northridge | 6.69 | LA - Centinela | 25.44 | 0.37 |
| 7 | 1995 | Kobe | 6.9 | Kakogawa | 26.4 | 0.34 |

## 5. ANALYTICAL MODELING OF STRUCTURES

Modeling of 2-D frames is carried out using IDASS [17] software. IDASS is an enhanced version of IDARC [18] which is able to model behavior of ductile connections and post fracture of welded connections [19]. Centerline models are used. Therefore, panel zone effects are not modeled. Global P-Δ effects are considered in analysis. Beams are modeled as elastic elements and nonlinear behavior of beams is included by nonlinear rotational springs at the end of beams.

Three degrading hysteresis rules are considered. The first model captures steel beam local buckling with 25% strength loss in each cycle. The second model represents Foutch and Shi model for weld fracture. This model does not include stiffness degradation for connection fracture. Hysteresis parameters are taken from the SAC study performed at Stanford University [20]. The third model represents Kunnath proposed model for connection fracture. This model include both stiffness and strength degradation due to fracture based on the results of SAC Joint Venture tests on welded connections [21]. To model brittle connections, every connection is assumed to be able to fracture.

Finally, an elastic perfectly plastic (EPP) hysteresis rule is modeled to compare the results of degrading models with a non-degrading one.

## 6. NONLINEAR STATIC ANALYSIS

*6.1 Lateral load distribution*

Using an appropriate lateral load pattern in evaluation of building behavior is one of the important steps. In fact, the shape of lateral loading presents how inertia forces are distributed during an earthquake. Two lateral load patterns were used here to consider two extreme cases affecting the structure behavior: (1) Inverted triangular load pattern and (2) Uniform load pattern.

*6.2 Calculation of Target Displacement*

FEMA440 estimates the target roof displacement using Eq. (1) as follows:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \;\;\&\;\; \mathrm{R} < \mathrm{R}_{\max} \tag{1}$$

Where, $T_e$ is the effective fundamental period in. $S_a$ is the amount of site response spectrum acceleration and $C$ coefficients are modification factors. Figures 7 to 10 show the resulting pushover curves. Calculated target displacements are included in Tables 2 and 3.

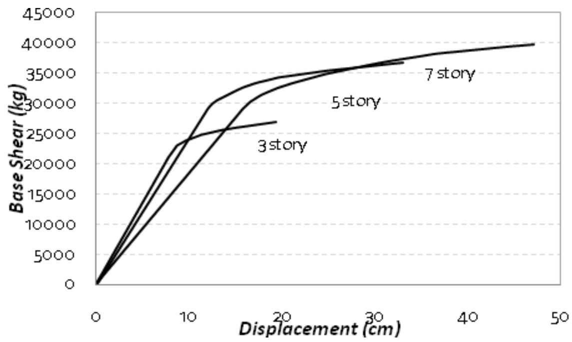



Figure 7. Pushover curves for 2-bay intermediate moment frames

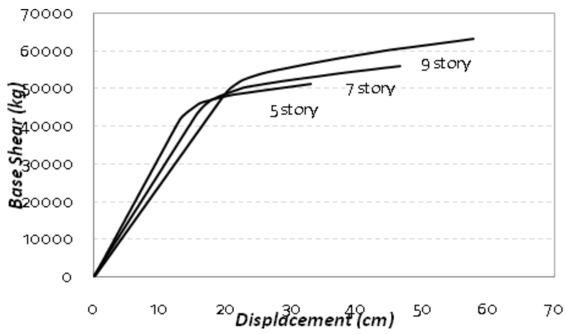



Figure 8. Pushover curves for 3-bay intermediate moment frames

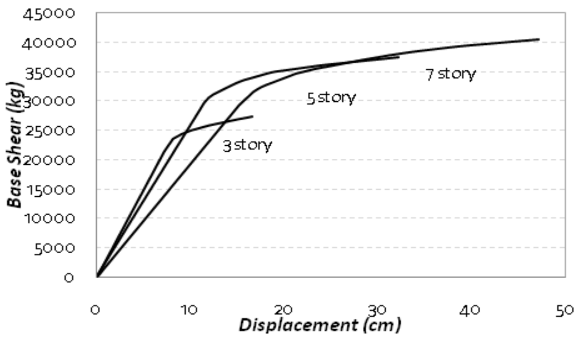



Figure 9. Pushover curves for 2-special moment frames

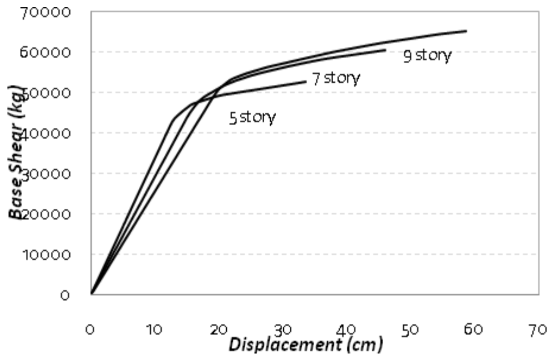



Figure 10. Pushover curves for 3-bay special moment frames

Table 2: Nonlinear static analysis results for intermediate moment frames

| 2-bay frames | | | |
|---|---|---|---|
| | 3 story frame | 5 story frame | 7 story frame |
| Triangular load | 14.91 | 21.03 | 31.01 |
| Uniform load | 14.76 | 20.88 | 30.91 |
| 3-bay frames | | | |
| | 5 story frame | 7 story frame | 9 story frame |
| Triangular load | 23.47 | 31.55 | 39.78 |
| Uniform load | 23.34 | 31.55 | 39.57 |

Table 3: Nonlinear static analysis results for special moment frames

| | | | |
|---|---|---|---|
| 2-bay frames | | | |
| | 3 story frame | 5 story frame | 7 story frame |
| Triangular load | 9.79 | 17.26 | 22.23 |
| Uniform load | 9.80 | 17.26 | 22.18 |
| 3-bay frames | | | |
| | 5 story frame | 7 story frame | 9 story frame |
| Triangular load | 16.14 | 22.11 | 26.88 |
| Uniform load | 16.19 | 22.14 | 26.83 |

## 7. NONLINEAR DYNAMIC ANALYSIS

In this study a set of seven earthquake records were used. Therefore, according to FEMA356, average roof displacements obtained from each record is considered as the roof displacement demands. Results of dynamic analysis are depicted in tables 4 to 7.

According to FEMA440, $C_2$ coefficient is calculated from the Eq. (2). For periods greater than 0.7 sec, $C_2$ may be assumed equal to 1.0.

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 \tag{2}$$

Here, $C_2$ is calculated from comparison of dynamic degrading and dynamic non-degrading analysis responses. Tables 4 to 7 summarize the results. Figures 11 and 12 also show mean error statistics of nonlinear dynamic analysis response values.

Table 4: Nonlinear static and dynamic results for 2-bay intermediate moment frames

| | Record number 1 | 2 | 3 | 4 | 5 | 6 | 7 | Average | $C_2$ coefficient |
|---|---|---|---|---|---|---|---|---|---|
| 3 story frame | | | | | | | | | |
| Bilinear | 8.79 | 10.21 | 11.96 | 10.67 | 19.35 | 7.95 | 14.83 | 11.96 | - |
| Kunnath | 9.09 | 10.19 | 11.96 | 9.73 | 20.22 | 7.95 | 9.58 | 11.25 | 0.94 |
| Foutch | 11.36 | 20.07 | 11.36 | 7.02 | 14.41 | 9.39 | 8.43 | 11.72 | 0.98 |
| Strength deg. | 11.79 | 19.88 | 11.48 | 8.23 | 15.19 | 10.22 | 8.57 | 12.19 | 1.02 |
| 5 story frame | | | | | | | | | |
| Bilinear | 16.71 | 16.99 | 20.40 | 21.89 | 16.13 | 15.77 | 16.28 | 17.75 | - |
| Kunnath | 17.27 | 19.20 | 22.45 | 26.04 | 19.91 | 17.17 | 16.23 | 19.76 | 1.11 |
| Foutch | 15.70 | 17.54 | 24.01 | 12.91 | 20.73 | 16.31 | 20.76 | 18.28 | 1.03 |
| Strength deg. | 15.87 | 17.32 | 23.52 | 14.60 | 20.87 | 17.01 | 23.73 | 18.99 | 1.07 |
| 7 story frame | | | | | | | | | |
| Bilinear | 26.09 | 21.08 | 26.85 | 16.08 | 30.66 | 14.71 | 25.55 | 23.01 | - |
| Kunnath | Failed | 21.84 | 27.28 | 16.00 | 39.37 | 16.81 | 21.11 | 23.75 | 1.03 |
| Foutch | 29.39 | 30.46 | 18.90 | 10.41 | 25.69 | 20.88 | 18.84 | 22.08 | 0.96 |
| Strength deg. | 30.27 | 29.59 | 20.28 | 12.57 | 26.70 | 27.03 | 24.23 | 24.39 | 1.06 |

Table 5: Nonlinear static and dynamic results for 3-bay intermediate moment frames

| | Record number | | | | | | | Average | $C_2$ coefficient |
|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | | |
| 5 story frame | | | | | | | | | |
| Bilinear | 19.05 | 16.69 | 16.13 | 19.94 | 19.05 | 13.89 | 20.75 | 17.93 | - |
| Kunnath | 19.81 | 16.89 | 23.98 | 27.43 | 22.28 | 14.12 | 17.22 | 20.24 | 1.13 |
| Foutch | 14.68 | 18.28 | 20.24 | 13.70 | 15.33 | 13.23 | 19.96 | 16.49 | 0.92 |
| Strength deg. | 15.05 | 17.57 | 20.76 | 13.15 | 15.57 | 14.06 | 20.51 | 16.67 | 0.93 |
| 7 story frame | | | | | | | | | |
| Bilinear | 28.88 | 23.29 | 23.42 | 37.29 | 26.37 | 19.76 | 24.61 | 26.24 | - |
| Kunnath | 35.15 | 29.26 | 23.42 | 35.97 | 39.17 | Failed | 19.86 | 30.48 | 1.16 |
| Foutch | 31.48 | 38.62 | 22.04 | 17.74 | 30.01 | 20.28 | 21.66 | 25.97 | 0.99 |
| Strength deg. | 29.91 | 35.43 | 21.87 | 20.18 | 31.09 | 24.34 | 28.21 | 27.29 | 1.04 |
| 9 story frame | | | | | | | | | |
| Bilinear | 28.32 | 25.86 | 21.92 | Failed | 38.91 | 22.61 | 28.63 | 27.71 | - |
| Kunnath | 42.60 | 29.60 | 22.43 | Failed | 54.03 | 28.47 | 46.79 | 37.34 | 1.35 |
| Foutch | 22.73 | 39.79 | 20.42 | 11.81 | 31.96 | 26.89 | 30.67 | 36.32 | 0.95 |
| Strength deg. | 25.11 | 37.08 | 20.41 | 12.45 | 32.79 | 39.56 | 32.35 | 28.54 | 1.03 |

Table 6: Nonlinear static and dynamic results for 2-bay special moment frames

| | Record number | | | | | | | Average | $C_2$ coefficient |
|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | | |
| 3 story frame | | | | | | | | | |
| Bilinear | 6.76 | 7.87 | 11.38 | 7.49 | 17.15 | 5.56 | 10.49 | 9.53 | - |
| Kunnath | 6.78 | 7.85 | 11.35 | 6.40 | 17.45 | 5.59 | 10.34 | 9.39 | 0.99 |
| Foutch | 7.62 | 9.94 | 10.82 | 5.67 | 14.45 | 9.87 | 6.97 | 9.33 | 0.98 |
| Strength deg. | 7.80 | 10.17 | 11.58 | 5.72 | 14.21 | 10.19 | 7.69 | 9.62 | 1.01 |
| 5 story frame | | | | | | | | | |
| Bilinear | 17.68 | 17.86 | 19.53 | 8.64 | 16.05 | 15.32 | 13.23 | 15.47 | - |
| Kunnath | 16.21 | 17.15 | Failed | 8.69 | 22.38 | 14.58 | 13.21 | 15.37 | 0.99 |
| Foutch | 13.21 | 20.09 | 16.70 | 8.03 | 13.92 | 13.30 | 16.54 | 14.54 | 0.94 |
| Strength deg. | 13.50 | 19.65 | 16.69 | 10.02 | 13.50 | 13.73 | 17.90 | 15.00 | 0.97 |
| 7 story frame | | | | | | | | | |
| Bilinear | 22.15 | 21.21 | 24.41 | 14.94 | 27.74 | 17.45 | 23.55 | 21.64 | - |
| Kunnath | 33.86 | 23.72 | 25.37 | 19.86 | 34.93 | 18.34 | 17.96 | 24.86 | 1.15 |
| Foutch | 24.64 | 29.70 | 9.05 | 9.86 | 23.68 | 16.59 | 21.29 | 19.25 | 0.89 |
| Strength deg. | 26.41 | 32.40 | 19.96 | 12.64 | 23.66 | 23.74 | 27.76 | 23.80 | 1.10 |

Table 7: Nonlinear static and dynamic results for 3-bay special moment frames

| | Record number | | | | | | | Average | $C_2$ coefficient |
|---|---|---|---|---|---|---|---|---|---|
| | **1** | **2** | **3** | **4** | **5** | **6** | **7** | | |
| 5 story frame | | | | | | | | | |
| Bilinear | 20.14 | 15.65 | 15.47 | 11.00 | 15.29 | 15.11 | 20.70 | 16.19 | - |
| Kunnath | 24.94 | 15.21 | 24.64 | 11.00 | 18.24 | 15.11 | 18.01 | 18.16 | 1.12 |
| Foutch | 15.55 | 20.01 | 17.58 | 11.79 | 11.82 | 14.30 | 16.55 | 15.38 | 0.95 |
| Strength deg. | 16.19 | 19.60 | 20.66 | 9.64 | 11.76 | 13.60 | 17.31 | 15.54 | 0.96 |
| 7 story frame | | | | | | | | | |
| Bilinear | 23.27 | 24.99 | 22.38 | 15.29 | 27.94 | Failed | 23.57 | 22.91 | - |
| Kunnath | 31.34 | 27.31 | 22.38 | 17.48 | Failed | 21.84 | 21.51 | 23.64 | 1.03 |
| Foutch | 25.95 | 32.93 | 23.21 | 15.58 | 22.20 | 16.70 | 17.38 | 21.99 | 0.96 |
| Strength deg. | 28.47 | 34.92 | 21.52 | 13.63 | 25.51 | 25.59 | 29.93 | 25.65 | 1.12 |
| 9 story frame | | | | | | | | | |
| Bilinear | 41.91 | 25.98 | 20.02 | 32.36 | 27.86 | 33.30 | 22.66 | 29.16 | - |
| Kunnath | 32.46 | 33.96 | 20.02 | 19.99 | Failed | 30.33 | 23.67 | 26.74 | 0.93 |
| Foutch | 22.96 | 39.04 | 23.06 | 16.68 | 39.54 | 28.95 | 27.68 | 28.28 | 0.97 |
| Strength deg. | 24.32 | 37.99 | 22.98 | 13.98 | 39.86 | 44.32 | 30.83 | 30.61 | 1.05 |

As can be seen from the figures, error values are almost independent of strength degradation type. However, the effect of stiffness degradation is more obvious especially for the case of special moment frames. The mean errors for strength degradation case are decreased by increase in fundamental period of the structures.

There is an acceptable correlation between static pushover maximum displacements and nonlinear dynamic values for Special frames. But errors are increased by increase in fundamental period of structure for the case of stiffness degradation. For Intermediate frames static pushover procedure underestimates displacement demands about 10% to 30% for degrading behavior.

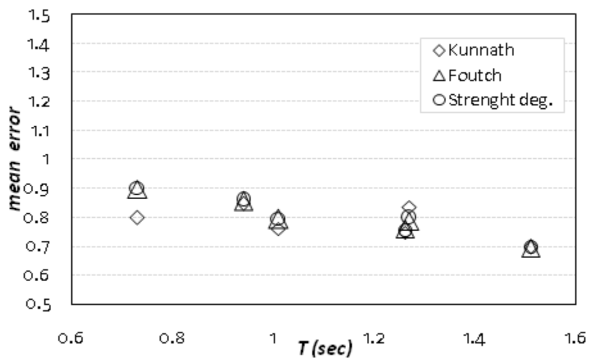



Figure 11. Mean errors for intermediate moment frames

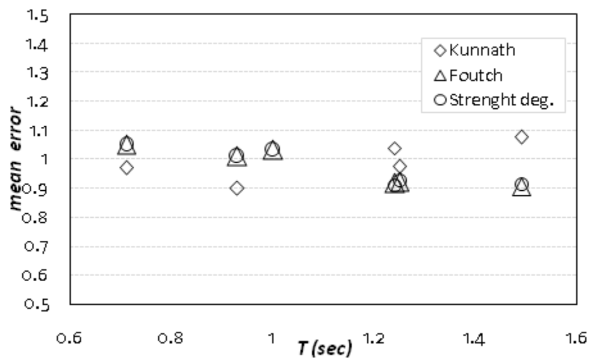



Figure 12. Mean errors for special moment frames

Figures 13 and 14 compare FEMA440 proposed $C_2$ values and calculated ones. According to FEMA440, the magnitude of $C_2$ for all frames are equal to 1.0.

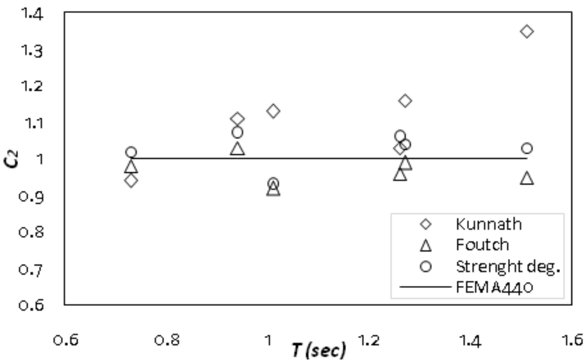



Figure 13. Comparison of $C_2$ values

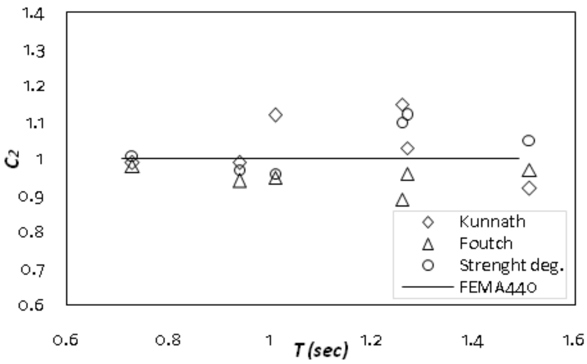



Figure 14. Comparison of $C_2$ values

It can be seen that increase in displacements due to strength degradation is limited to less than 10%. Influence of stiffness degradation on displacement demands is more sensible for Intermediate frames, since stiffness degradation led to increase in maximum displacements up to 35%. However, this is moderate for special moment frames (less than 15%). The average values of calculated $C_2$ coefficient is shown in Table 6.

Table 6: Average values of $C_2$

| | **Kunnath** | **Foutch** | **Strength deg.** |
|---|---|---|---|
| Special | 1.06 | 0.97 | 1.04 |
| Intermediate | 1.12 | 0.95 | 1.03 |

## 8. CONCLUSION

This study shows the effect of strength and stiffness degradation on intermediate and special WSMFs. From the results, the following conclusions can be drawn:

- Strength degradation has no significant effect on displacement demands. However, values of $C_2$ for models including stiffness degradation are more than the others.
- In general, static nonlinear analysis estimates maximum displacements with an acceptable accuracy for degrading systems.
- Degradation has less effect on special moment steel frames than intermediate frames.

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