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Full Length Research Paper

Evaluation of various methods of FEMA356 compare to FEMA440

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The purpose of this paper is to evaluate the performance-based procedures in the FEMA356 which is used for vulnerability assessment of existing buildings. The FEMA 356 contained four analysis methods. These methods are linear and nonlinear static / dynamic analysis. For this evaluation, two programs are studied (1) the Displacement Coefficient Method and (2) the buildings designed by UBC97. In this study, several special steel moment-resisting buildings are designed according to the UBC97 requirements and their vulnerability is assessed. These buildings are from 1 story to 20 stories. The designs of these buildings perform by AISC requirement. The seismic load for design of these buildings extract from UBC 97. The UBC 97 contained two analysis methods. These methods are linear static / dynamic analysis. Analytical results show that some columns do not satisfy the life safety performance in the design hazard level. Moreover, the target displacement estimated by the Displacement Coefficient Method is larger than the maximum displacement calculated by nonlinear dynamic analysis.

Key words: FEMA356, UBC97, performance-based procedures, Displacement Coefficient Method.

INTRODUCTION

In UBC97, design criteria based on multiple coefficients were supposed as more of the ground motion and response phenomena became known. Most of these coefficients are from good engineering judgment and rely on physical concepts and equations. In most aspects designs were force-based, and required providing adequate strength to all elements of the lateral load resisting system. Nowadays the UBC97 is used for seismic design of new buildings in Iran (UBC97, 2000). FEMA356 is used for vulnerability assessment of existing buildings (FEMA, 2000). This Guideline recommends four analysis procedures to estimate seismic demands. The first one is the linear static procedure and the second one is the linear dynamic procedure. These two methods are force-based. The third method is the nonlinear static procedure. This procedure uses the Displacement Coefficient Method in which the modeled Structure is displaced to a target displacement by means of a pushover analysis. The fourth method is the nonlinear dynamic procedure. The third and fourth methods are displacement-based.

In this paper, the performance-based procedures in the FEMA356 are assessed. For this assessment, the UBC97 is used as a benchmark.

DESCRIPTION OF BUILDINGS

The sample buildings are of 1-story to 20-story. These buildings have special steel moment frames as the lateral resisting system.

The sample buildings are designed according to the UBC97 provisions. For design of frames, the linear static analysis is used for all sample buildings, except 15 and 20 story buildings, for which the linear static analysis is not adequate. In these cases, a linear dynamic analysis should be used to specify and distribute the seismic design forces.

ANALYSIS PROCEDURES

A vulnerability assessment objective shall be selected for the building. In UBC97, the goal of design is life safety performance for design earthquake. Therefore, the basic safety objective is adopted for vulnerability assessment of the sample buildings. The basic safety objective is defined as life safety building performance level for the earthquake hazard level 1.

The FEMA356 suggests four analysis procedures to estimate the seismic demands. Of these four methods, the linear and nonlinear dynamic procedures, and the nonlinear static procedure as well are used in this study.

Linear dynamic analysis

In the linear dynamic procedure, the design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly elastic dynamic analysis in compliance with the requirements of the FEMA356. This procedure includes the response spectrum method and the time history method. The response spectrum method uses peak modal responses calculated from the dynamic analysis for a mathematical model. Only those modes contributing significantly to the response need to be considered. Modal responses are combined using rational methods to estimate total building response quantities. The time-history method involves a time-step-by-time-step computation of building response, using recorded or synthetic earthquake records as base motion input. However, the response spectrum method was used for the linear dynamic procedure. In this method, the value of the usage ratio is calculated as following (FEMA, 2000):

a. Beam: The value of the usage ratio is defined as the ratio of the DCR_m to m-factor.

$$U_r = \frac{M_U}{m \times M_{CE}} = \frac{DCR_m}{m} \quad (1)$$

Where:
DCR_m is defined as the ratio of internal force to the strength of beam.
m is partial ductility coefficient(m-factor). This parameter is given in Table 2 in FEMA356.
M_U is the bending moment in the member, calculated in accordance with the linear analysis.
M_{CE} is the expected flexural strength of beam components and shall be determined using equations for design strength, given in AISC (1997) *Seismic Provisions*, except that the reduction factor of strength, ϕ, shall be taken as 1.0 and $1.1F_y$ shall be substituted for the yield stress (F_y is the lower-bound strength).

b. Column: For steel columns under combined axial compression and bending stress, where the axial column load is less than 50% of the lower-bound axial column strength, P_{CL}, the column shall be considered as deformation-controlled for flexural behavior and force controlled for compressive behavior. In this case, the value of the usage ratio shall be evaluated by equation 5.12 to 5.14 in the FEMA356.

But steel columns with axial compressive forces exceeding 50% of the lower-bound axial compressive strength, P_{CL}, shall be considered as force-controlled for both axial loads and flexure and the value of the usage ratio shall be evaluated using equation 5.15 and 5.16 in the FEMA356.
This ratio was calculated for beams and columns based on the results of linear dynamic analyses and are shown in this “linear dynamic analysis” part of the work.

Nonlinear static analysis

In the nonlinear static method the internal forces and deformations are evaluated for the corresponding target displacement. The target displacement intends to represent the maximum displacement that the structure can reach during the design earthquake.

The Displacement Coefficient Method is the primary nonlinear static procedure presented in FEMA356. This approach modifies the linear elastic response of an equivalent SDOF system by multiplying it by a series of coefficients from C_0 to C_3 to estimate the maximum global displacement of the building, which is termed the target displacement (FEMA 440, 2005). Target displacements are calculated for all sample buildings and shown in Table 1.

In nonlinear static method, the value of the usage ratio is defined as the ratio of the deformation demand to deformation capacity (FEMA, 2000).

$$U_r = \frac{\theta}{\theta_{LS}} \quad (2)$$

Where:
θ is the deformation demand. This parameter is obtained from the nonlinear static analysis.
θ_{LS} is deformation capacity for life safety performance. This parameter is calculated by addition of the yield rotation to the value of Table 3 in the FEMA356.

This ratio was calculated for beams and columns based on the results of nonlinear static analysis and are shown in “Nonlinear static analysis” part of this work.

Nonlinear dynamic analysis

For development of time-histories, Abhar, Zanjan, Lahijan records are used. These real time-histories were recorded on soil type III.

The selected time-histories should be modified to be closer to the design ground motion conditions. The requirements in the FEMA356 are as following:

Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time history components that shall be selected and scaled from not less than three recorded events. The motions shall be scaled such that the values of response spectrum of earthquake partly match to the 5 percent-damped spectrum of the design-basis earthquake for periods from 0.1T second to 3T seconds (T is fundamental period of the building). The parameter of interest shall be calculated for each time history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design (FEMA, 2000).

In this study, the motions are scaled by the FEMA356 requirements. The scale factor for Abhar, Zanjan and Lahijan earthquake are shown in Table 2 (Keyvani, 2008).

All sample buildings are analyzed for any three records and the maximum displacement on the roof of buildings is extracted. For any building, the maximum of these three values that have been calculated are written in Table 3 (Keyvani, 2006).

Table 1. Target displacement.

Story	Target displacement
1 story	0.10
2 story	0.15
3 story	0.20
5 story	0.28
10 story	0.55
15 story	0.88
20 story	1.16

Table 2. Scale factor.

Place	Scale factor
Abhar	3.69
Zanjan	3.96
Lahijan	5.54

Table 3. Maximum displacement.

Story	Maximum displacement
1 story	0.11
2 story	0.15
3 story	0.17
5 story	0.25
10 story	0.31
15 story	0.50
20 story	0.60

In nonlinear dynamic method, the calculation of the usage ratio is the same as previous method. This ratio was calculated for beams and columns based on the results of nonlinear dynamic analysis and are shown in "nonlinear dynamic analysis and evaluation of that by linear static and dynamic analysis of UBC97" part of the work.

RESULTS AND DISCUSSION

The usage ratios of columns and beams demonstrate that some lower columns of buildings do not satisfy life safety performance in the design hazard level while all beams of buildings satisfy that.

Linear dynamic analysis

Here, the linear dynamic analysis of the FEMA356 is evaluated. For this evaluation, the linear static and spectral analyses of UBC97 are used. This is done by comparison of the UBC97 and FEMA356 are used. As seen in this result:

A. In lower columns of building:

The linear static analysis and linear spectral analysis of UBC97 is located below the linear dynamic analysis of FEMA356.

B. In upper columns of building:

The linear static analysis and linear spectral analysis of UBC97 is appropriately matched to the linear dynamic analysis of FEMA356.

This result arises from difference of concepts of two codes (UBC97 and FEMA356). The UBC97 uses the behavior coefficient (R) to bring the nonlinear behavior in analysis. While the FEMA356 uses the partial ductility coefficient (m-factor) for this purpose.

The behavior coefficient is constant for all member of an individual building. But m-factor depends on axial forces of members (Equation 3) (FEMA, 2000).

$$m = 8\left(1 - 1.7\frac{p}{p_{CL}}\right) \quad (3)$$

Where:
P is axial force in the member calculated in accordance with the linear analysis.
P_{CL} is the effective design strength or the lower-bound axial compressive strength of column components and is calculated in accordance with AISC (1997) Seismic Provisions, taking ϕ =1.0 and using the lower-bound strength, F_y, for yield strength.

Therefore in lower columns, where the amounts of axial forces are high, the m-factor is low and consequently the UBC97 is located below the FEMA356.

Nonlinear static analysis

Here, the nonlinear static analysis of the FEMA356 is evaluated. For this evaluation, the linear spectral analysis of UBC97 is used. As seen in this result:

A. In lower columns of frames:

The linear spectral analysis of UBC97 is located below the nonlinear static analysis of FEMA356.

B. In upper columns of frames:

The linear static analysis and linear spectral analysis of UBC97 is located above the nonlinear static analysis of FEMA356.

As stated before, the behavior coefficient is constant for all member of an individual building. But the capacity of

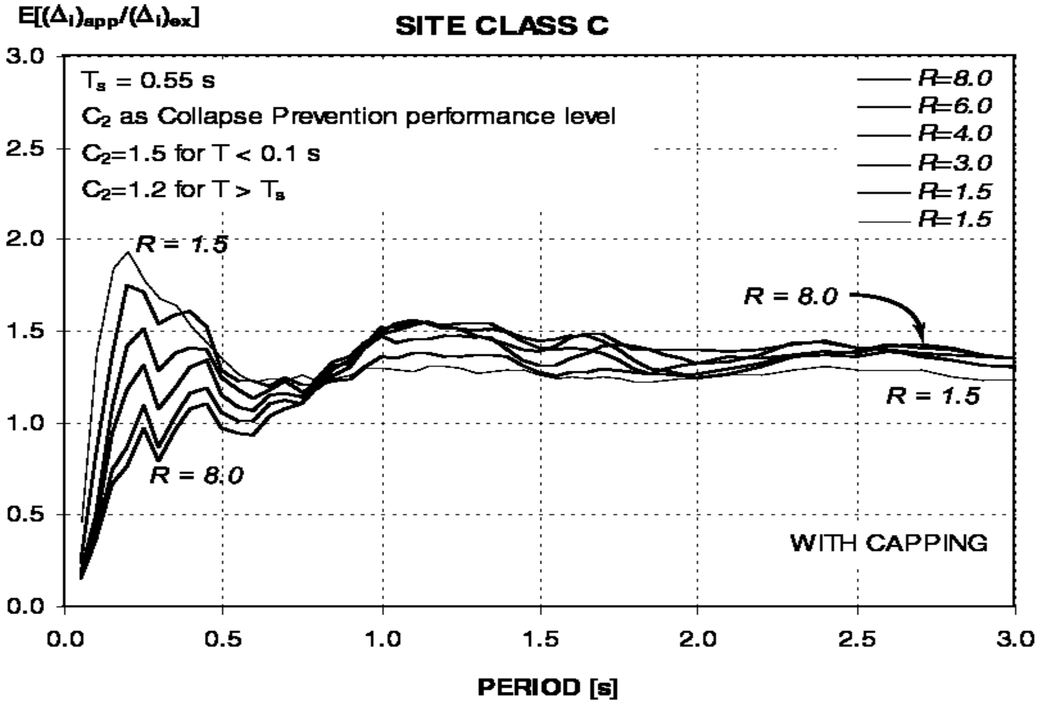

Figure 1. The mean error statistics associated with *C*1 and *C*2 assuming a Collapse Prevention performance level in accordance with FEMA 356 for stiffness and strength (SSD) degrading systems (FEMA 440, 2005).

nonlinear behavior depends on axial forces of members (Equation 4) (FEMA, 2000).

$$\theta_P = 7\left(1 - 1.7\frac{P}{P_{CL}}\right)\theta_y \quad (4)$$

Where

θ_y is yield rotation of the member.

θ_p is plastic rotation capacity of the member.

Therefore in lower columns of frames, where the amounts of axial forces are high, the plastic rotation capacity is low and consequently the UBC97 is located below the FEMA356.

Nonlinear dynamic analysis and evaluation of that by linear static and dynamic analysis of UBC97

Here, the nonlinear dynamic analysis of the FEMA356 is evaluated. For this evaluation, the linear spectral analysis of UBC97 is used. The result of this part of the work is the same as that of nonlinear static analysis.

EVALUATION OF TARGET DISPLACEMENT

There are two options for use nonlinear static procedures. Those are the Capacity-Spectrum Method is documented in ATC-40 and the Displacement Coefficient Method which is presented in FEMA 356. Both approaches use nonlinear static analysis to estimate the lateral force-deformation characteristics of the structure.

The FEMA 440 is the principal product of the ATC-55 Project. This report evaluates both current procedures by a series of nonlinear single-degree-of-freedom oscillators of varying period, strength, and hysteretic behavior. These oscillators were subjected to ground motions representing different site soil conditions. The resulting database of approximately 180,000 predictions of maximum displacements was used as a benchmark to judge the accuracy of the approximate nonlinear static procedures. This was accomplished by comparing the estimates for each oscillator from both nonlinear static procedures to the results of the nonlinear response history analyses. Differences in the two estimates were compiled and compared in a statistical study (Keyvani, 2006).

FEMA 440 summarizes the results of studies to assess the ability of the Displacement Coefficient Method to estimate the maximum displacement of inelastic structural models.

For example, Figure 1 presents mean errors calculated from the ratio of the displacements computed using *C*1 and *C*2 as determined from FEMA 356 to maximum displacements computed with nonlinear response history

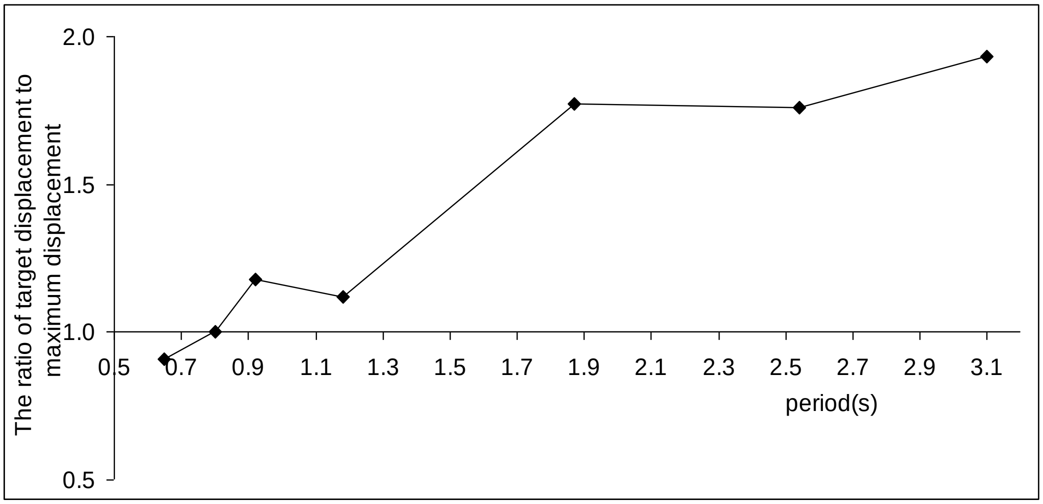

Figure 2. The ratio of target displacement to maximum displacement in roof of buildings.

analyses for the stiffness and strength degrading systems. Results in this case correspond to site class C. This figure shows that the target displacement is overestimated when the period is larger than 0.5 s (FEMA 440, 2005).

In this paper, the buildings with special steel moment frames have been studied. For these buildings, ratio of the target displacement (values in Table 1) to maximum displacement (values in Table 3) is calculated and shown in Figure 2. This figure demonstrates that the nonlinear static procedure introduced in the FEMA356 over-estimates the target displacement for buildings which have long and medium periods.

Conclusions

Results of this study show that a few numbers of lower columns of the selected frames do not satisfy life safety performance for the design hazard level. Therefore, the UBC97 does not match to the FEMA356 in life safety performance for the design hazard level (Keyvani, 2008).

This result is important for buildings designed by UBC97. According to this result, the buildings which are being established now should be rehabilitated. Comparison between the nonlinear and linear analysis shows that the results of these two analyses are completely different. So the linear analyses are not reliable for vulnerability assessment of building with moment resisting frame. Moreover, Results of the nonlinear static and dynamic analyses show that the Displacement Coefficient Method overestimates target displacement. This result is also mentioned in FEMA440. But all analyses used in the FEMA440 are obtained from one degree freedom models (FEMA 440, 2005).

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