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Full Length Research

Probabilistic assessment of performance point in structures and examination of the instabilities

***Mehran Satrapi, Abbas F. Jobrani and Majid Amanpour**

Department of Civil Engineering, Allameh Tabataba'i University,Tehran, Iran.

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The main goal of the performance based design of structures is to rationally predict the structures' performance during earthquakes which may occur during the lifetime of the structure. In this sort of design, a specific displacement is defined as target displacement and the structure is subjected to a force in order to reach this target displacement. This design process includes uncertainties in loading, materials and analysis methods of the performance point. Therefore, statistical and probabilistic analysis should be considered. In this paper, uncertainty sources for determining the performance point are defined and then the procedures suggested in the codes are introduced. In the next step, an appropriate probability distribution function is defined for uncertainty parameters and finally the performance point of the structure is determined regarding these parameters in accordance with the codes. In addition, the sensitivity of the performance point with respect to the mentioned parameters is investigated. Results indicate that sensitivity of the performance point to geometric characteristics is of great importance and other parameters such as dead and live load stand in the second level in terms of sensitivity. An appropriate lateral loading pattern with the least uncertainty is also proposed for buildings.

Key words: Performance level, performance point, probabilistic design, uncertainty, sensitivity analysis.

INTRODUCTION

The design process is divided into three major parts: Loading, analyzing and final designing. In the first part (loading), the effects of environment on the structure like lateral and gravity loads are considered. The real nature of these loads makes them unavoidably accidental. Simulation of the structure's real behaviour under the imposed loads is the main purpose of such modelling. Due to the complexity of real relations, some simplifying assumptions are generally included in the modelling process and this imposes more uncertainty to the process. On the other hand, the human mistakes would be added to these circumstances.

Therefore, this would make the probabilistic assessment of civil engineering processes inevitable. In recent years, researchers have moved forward in designing structures with high loading resistance during earthquake. In other words studies in designing trustable structures confronting earthquake loads are in progress. These improvements include changing designing methods, that is, force method to the displacement method which is called performance based design. This method is based on a way of accepting displacement based on reliable scientific codes such as FEMA (Federal Emergency Management Agency) 365(2000), FEMA 273(1997), and ATC(Applied Technology Council) 40(1997). In scientific literature, four performance levels are defined in relation with building performance based design of structures which are introduced as serviceable level, Immediate Occupancy (IO), Life safety (LS) and collapse prevention (CP), so that each one has its specific characteristics.

The principal criterion in considering the structural status of buildings is their target displacement.

*Corresponding author. E-mail: satrapi.mehran@gmail.com.

Therefore, determining target displacement is the most important part of performance based designing.

The serviceability level is the level which would not lead to any damage in the structure. In the immediate occupancy level, there would a limited damage in structural elements. In life safety level, the structure may experience huge damage on structural elements and the collapse prevention would occur when a structure approaches a local or global unstable condition. The aforementioned levels are based on the target displacement point, therefore finding the target point or the performance displacement is the most important part of performance based design process.

There are many direct or indirect parameters affecting the determination of this point. A great number of these parameters externally affect the process and some of them internally influence the process.

Factors such as the pattern of lateral load of earthquake, soil characteristics, design accelerating, dead and live loads are categorized as external factors and the behaviour of structural elements; connections and damping of the structure are the effective internal factors in determining target displacement. Furthermore, soil and structure interactions could be placed in both groups.

The steps and procedures of determining the performance point are cited in FEMA365 (2000) and ATC40 (1997). These references show the uncertainties which exist in this method. Even selecting the procedure to define the performance point has some uncertainties. Therefore, identifying and evaluating these resources of uncertainty in defining the performance point would be important due to their importance in the process of designing based on the performance of the structure.

Sensitivity analysis would be helpful in these cases. Therefore, after determining the uncertainty resources, a sensitivity analysis is utilized. Finding the sensitivity of performance point function with respect to the parameters may be useful for distinguishing the sensitive parameters from insensitive ones in reliability process. Since the performance based design procedure is an innovative idea, many researchers have done research on this case. Chopra (1992), Fajfar (1996) and Krawinkler (1998), presented new performance based design process. It could be observed that all researchers assumed that the whole existing parameters related, and many of the researchers barely focused on the development and innovation of the performance method. Otherwise if the uncertainties were surveyed probabi-listically, the extent of researches on the sensitive parameters could be extended and also limited on the insensitive parameters. The main goal of this study is to identify the resources of uncertainties and finding sensitivity of the performance point related to them so as to be more confident during designing process.

UNCERTAINTIES IN CIVIL ENGINEERING AND ITS RESOURCES

Designing process in civil engineering suffers from a variety of uncertainties. Some of their hidden characteristics are transparently distinguishable and some of them are not. Uncertainties in civil engineering could be divided into two groups; stochastic and uncertainty in realizing systems and their constituents. The first group has a probabilistic nature while the other group relied on human knowledge of the entire system and its components' behaviour (William and Bulleit, 2008). The most important resource of uncertainties in civil engineering could be divided into 5 groups.

Uncertainty in loading

The probabilistic nature of loads subdivides them into two large groups; the gravity loads and lateral loads. The gravity loads could be divided into three parts; dead loads, permanent live loads and transient live loads. Dead loads are constant during the lifetime of structures with the least uncertainty among other loads. Permanent live loads are constant in time intervals, the time intervals may change along with the change in the application of the structure. The transient live loads are applied in specific times during the structure's life time.

Due to complexity in choosing the models for distributing live loads, these loads have more uncertainty compared to dead loads (Ranganathan, 2000).

The lateral loads belong to those groups which their effects on the structures are of great importance and could not be neglected. These loads because of their risky and dynamic nature may lead to instability and demolition of the structure. Wind load and earthquake load are of these kinds. Figure 1 shows the variation of loads with respect to the lifetime of the structure (Nowak and Collins, 2000).

Generally, the most elaborate load imposed to the structure is the earthquake load. Therefore many uncertainties exist in this case.

The time of earthquake occurrence, the distance between the structure and the earthquake occurring place, the earthquake intensity, shaking duration, the focal depth and etc are of some problems relating to the external effects of uncertainties in earthquake loads (uncertainties in earthquake engineering). The internal effects of uncertainty in earthquake loads on the structure may comprise the behaviour of elements against the earthquake reciprocal loads, the behaviour of joints, damping, fatigue and the response of whole system to the imposed vibration (uncertainties in earthquake engineering). The earthquake loads imposed to the

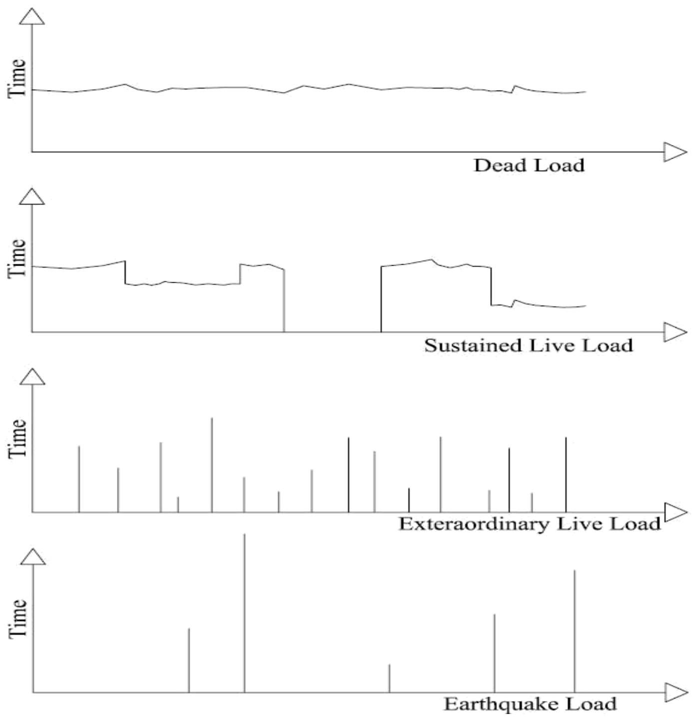

Figure 1. Variation of different loads during the life time.

structure are a combination of the dead loads and live loads, therefore the uncertainties existing in the earthquake load are greater than other loads. The live permanent loads stand in the second place and the dead loads stand in the third place.

Uncertainty in resistance

The structural designers define the characteristics of materials and manufacturers that produce such materials with specified standards. Some problems such as variation in strength of the materials, the member's dimensions or inherent changes in used equations may cause the strength of the structural elements not to be equal to their nominal strength. Therefore, the uncertainty in the strength of materials would be inevitable.

Uncertainty in modelling

Responses of a structure to the applied loads corresponding to what happened in reality are the most important expectation of modelling process. While in modelling, there are some uncertainties which relate to a specific part of designing process. Regarding the expensive methods of explicit dynamic analysis, such as nonlinear dynamic analysis, linear static or nonlinear finite element analysis can be used. Applying such analysis would not always be profitable and some problems such as buckling, local and progressive failure would also occur causing the real behaviour of the structure not to be reflected in the model. Therefore, this part would contain uncertainties due to the simplifying assumptions in modelling phase.

Uncertainty in selecting the designing codes

Design of structures is generally based on specific designing codes with different methods. There are totally three major methods used in designing structures: Allowable stress, ultimate stress and plastic method. There are specific coefficients for loads and strengths in each code according to the method selected. Utilizing different codes for designing structures would not lead to similar results. Therefore, selecting a specific design code is also one of the uncertainties in designing process.

Human error

Generally, human errors are a part of engineering science expressed as performance and self awareness errors known as sources of uncertainties (Nowak and Collins, 2000). In order to minimize this sort of error, employing experts for checking computations, excellent supervision and accurate construction process is necessary.

DETERMINATIONS OF PERFORMANCE POINT AND UNCERTAINTIES

During the process of the performance based design method, the structure is pulled by a set of lateral loads. By increasing the lateral displacement, forces inside structural members increases, making some points of the structure exceed the yield limit and this matter leads to formation of plastic hinges. According to the performance level selected for the structure, the structure must resist a certain lateral displacement without increasing the deflection of members over the allowable limits. Target displacement method in FEMA 356 and the spectrum capacity method introduced in ATC 40 are the principal methods introduced for finding the performance point.

In FEMA 356 method, at first, a nonlinear analysis is performed and after that roof displacement versus base shear is plotted. This curve is the capacity curve of the structure.

The performance point could be defined by applying this curve along with the help of a series of coefficients. The mathematical form of the problem could be shown in Equation (1):

$$ {}_{T} = C_0C_1C_2C_3 \left(\frac{TE^{2}}{4\pi_{2}}\right)S_A \qquad (1) $$

where, C_0 is the relative displacement of the roof with respect to the spectral displacement; C_1 describes the ratio of plastic displacement to the elastic displacement; C_2 coefficient shows the stiffness reduction effects of the members and is C_3 indicates the P- effect; T_e is the main period and S_a is the spectral acceleration of the structure. Finally, the target displacement, $_t$ is obtained using Equation (1).

In ATC 40 method, first of all, the capacity and demand curve is plotted in the same format (ADRS). Afterwards by the use of a series of relations, the elastic spectra curve (on demand spectrum) is replaced with the non-elastic spectra curve. This stage would be accompanied by trial and errors leading to a point where elastic and non-elastic spectra would meet. The obtained point is the performance point.

An advantage of FEMA356 is its simplicity and its repetition free nature. The capability of ATC40 procedure relies on its plotting benefits.

The common part in both of these two methods is determining the response curve of the structure or the capacity curve, so that uncertainties are similar in this part.

The C_2, C_1 and C_0 coefficients introduced in Equation (1) are dependent on the behaviour and characteristics of the structure. Despite the existence of the calculation based relations for attaining this coefficient, some tables are tabulated in FEMA 356. By applying these tables, these 3 parameters would subsequently become certain parameters, and the rest of parameters would be still dependent on analyzing method and the spectral displacement. In ATC40 method, uncertainties are located in the section of defining capacity curve and the uncertainties relating to demand spectrum curve also exist. On the other hand, this procedure involves trial and error process and this matter leads to more uncertainties. This problem would worsen considering the convergence conditions (the difference between the attained spectral displacement and the assumed displacement should be less than 5%).

Some uncertainties in defining the performance point are shown in Table 1. These uncertainties have been considered in three groups: The capacity structure curve, the FEMA 365 and ATC 40 method. Some of these methods directly and some indirectly influence the designing process. Some variables are related to each other and are statistically dependant. For example, the period of the structure is affected by the mass and stiffness of the structure and the stiffness are dependent on cross sectional areas of structural elements. Therefore, the table could be more summarized. One can refer to the studies done in references (Ranganathan, 2000; Nowak and Collins, 2000; Cardoso et al., 2007; Cheng and Lib, 2007; Cheng, 2007; Huh and Haldar, 2002; Cornell et al., 2001; Bilal, 1998) for selecting the characteristics of the probability functions of these variables.

In this paper, only a limited group of variables in Table 1 marked by star are considered and their variation effects are investigated and the rest of variables are neglected so as to avoid complexity in the problem. For considering these effects, the variables in Table 2 are applied. In this table, the proposed characteristics of variables are attained from references (Ranganathan, 2000; Nowak and Collins, 2000; Cardoso et al., 2007; Cheng and Lib, 2007; Cheng, 2007; Huh and Haldar, 2002; Cornell et al., 2001; Bilal, 1998). As could be observed in Table 2, the references do not propose the same characteristics, and this matter leads to more ambiguous analysis. Finally, in Table 3, the statistical characteristics of random variables are selected. Applying normal or lognormal function is common when the type of probability function is unknown or sceptical (Bilal, 1998).

GENERAL ANALYSIS METHODS FOR EVALUATING THE STRUCTURE PERFORMANCE

Determination of target displacement is the most important part in performance method. Generally, the designing codes have proposed two analysis methods; linear elastic and nonlinear dynamic analysis for calculating the target displacement. Nonlinear static method is of less accuracy compared to nonlinear dynamic analysis. Even the best static nonlinear analysis procedure is not as accurate as dynamic ones in fulfilling the earthquake requirements. In spite of aforementioned subjects, applying nonlinear static analysis method in performance based design method is of great popularity. The reason is that design codes generally use the maximum response from three accelerometers or the mean of seven of them (Iranian code, 2800, 1999).

Uncertainty always exists according to selection of accelerometer, scaling, etc. Therefore the nonlinear static analysis has become an acceptable method in seismic analysis in recent years. Promptly acceptance of this method by designer is due to its simplicity and capability

Table 1. Uncertainty resources in performance point prediction for different methods.

ATC40	FEMA356	Capacity curve
Damping	Main period	Dead load*
Period	Effective stiffness	Live load*
Design spectrum	Bilinear approximation of capacity curve	Contribution percentage of live load during the earthquake
Bilinear approximation of capacity curve	Determination of C_0	Yielding and ultimate stress*
Determination of effective damping factor	Determination of C_1	Young’s module*
Hysteresis curve	Determination of C_2	Sectional areas of the members*(geometry)
Convergence condition control	Determination of C_3	Type of connections and the rigidness
	Design curve	Period
	Damping	Mass and stiffness distribution
		damping
		Effects of the modes shapes
		Non structural members
		Lateral load pattern in pushover analysis*
		Analysis method*
		Model of the structural member separately*
		Soil and structure interactions
		$P-$effect

Table 2. Random variables characteristics.

	Distribution function	Random variable	Distribution function	Coefficient of variation (C.O.V)	Reference
	Normal		Normal	0.1	Cardoso et al. (2007)
1	Normal	Dead load	Normal	0.08	Nowak and Collins (2000)
	Normal		Normal	0.12	Ranganathan (2000)
	Lognormal		Lognormal	0.36	Cardoso et al. (2007)
2	Lognormal	Live load	Lognormal	0.252	Ranganathan (2000)
	Lognormal		Lognormal	0.30	Nowak and Collins2000)
	Normal		Normal	0.05	Cardoso et al. (2007)
	Lognormal		Lognormal	0.10	Huh and Haldar (2002)
3	Uniform	Young’s module	Uniform	0.06	Cornell et al. (2001)
	Lognormal		Lognormal	0.06	Bilal (1998)
	Normal		Normal	0.091	Ranganathan (2000)
	Normal		Normal	0.1	Cardoso et al. (2007)
4	Normal	Yield stress	Norma	0.05	Cheng and Lib (2007)
	Normal		Normal	0.102	Huh and Haldar (2002)
	Lognormal		Lognormal	0.076	Ranganathan (2000)
	Normal		Normal	0.03	Cornell et al. (2001)
5	Lognormal	Geometry	Lognormal	0.10	Cheng (2007)
	Normal		Normal	0.04	Nowak and Collins (2000)

Table 3. Random variables and their information that have been used

Random variable	Distribution function	C.O.V
Dead load	Normal	0.10
Live load	Lognormal	0.35
Young's module	Normal	0.05
Yield stress	Lognormal	0.10
Geometry	Normal	0.04

of describing the natural behaviour of structure in nonlinear zones. Actually by the help of this method, the damaging zone, the progressive path of damage and the weak point of the structure could be defined. The principle of this method is that a nonlinear mathematical model of the structure under a lateral load pattern and this load increases with a fixed trend until the structure reaches an expected target displacement. This analysis is called "push over analysis".

The pattern which lateral forces distribute along the height of the structure is extremely complicated during earthquake shaking. The pattern of load distribution in the process of nonlinear static analysis has a remarkable effect on nonlinear behaviour of the structure, estimation of force requirement, system's displacement and structural members. In fact, the lateral load pattern shows the inertia forces applied to the structure arisen from ground motion.

In designing viewpoint, the distribution of lateral loads should be located in a way to prepare critical conditions on the structure. If the structure is still in linear elastic range, the distribution of lateral forces is dependent on frequency content, amplitude of vibration, the frequencies and the shape of the structural modes. If the structure has a nonlinear behaviour, the distribution of lateral forces will be dependent on local or general yielding of structural elements making the problem more complicated. The distribution of lateral loads changes as time passes and it may enter the nonlinear range in which the stiffness and the dynamic characteristics of structure vary. Generally, the lateral load pattern is divided into two types of variant and constant patterns (Antoniou and Pinho, 2004). In the state of constant lateral load pattern, the lateral loads are assumed to be constant and unchangeable. This assumption is unreal and approximate and changes in distribution of inertial forces which are caused by the variation of stiffness arisen from nonlinear behaviour of the structure could not be considered in the analysis. In order to prevent complexity of methods and because one of the aims of this research is evaluation of lateral load pattern on structures, therefore such approximation is accepted and we use constant pattern of lateral load. Some patterns of lateral loads which have been suggested by researchers, regulations and codes are constant lateral loads, lateral load of inverted triangle explained in Iranian 2800 code (1999) and UBC(American Seismic Building Code)97(1997); the lateral load of upside down triangle suggested in FEMA356 code (2000), seismic rehabilitation of building code 360 (2006) and the lateral load on the basis of structural mode. The typical procedure which is applied for studying and investigating the lateral load method of overload analysis is based on the differences between the results obtained and the results of nonlinear dynamic analysis of time history. In fact, any of the lateral load patterns which lead to closer results with respect to the nonlinear time history dynamic analysis could be introduced as a suitable lateral load pattern (Antoniou and Pinho, 2004). But since large amount of uncertainties exist in the nonlinear dynamic analysis such as the number and the type of accelerograms, the frequency, the distance from fault, etc. the judgment in this research is based on the least coefficient of variation in the performance point obtained from different lateral load patterns. Therefore, each pattern having the least scattering coefficient could be more suitable. In this research three lateral load patterns such as uniform, triangular distribution and the distribution based on the first mode of vibration of structures have been applied. Figure 2 indicates these distributions.

MODELLING

It is indeed preferable to apply the most accurate analysis and modelling to achieve better results and decrease the uncertainties and investigate the behaviour of the structure during earthquake. To reach this aim, modelling the behaviour of structural elements is done prior to linear design of the structure. In order to obtain the exact behaviour of rigid steel frames, at first samples of connections were modelled using ANSYS software and the behaviour of these joints (connections) in linear and nonlinear range were investigated. In order to extract moment-rotation relations, a fixed joint of a moment frame was used (in laboratory) (Abedi, 2000). Also in this paper, for extracting the moment-rotation relations, the

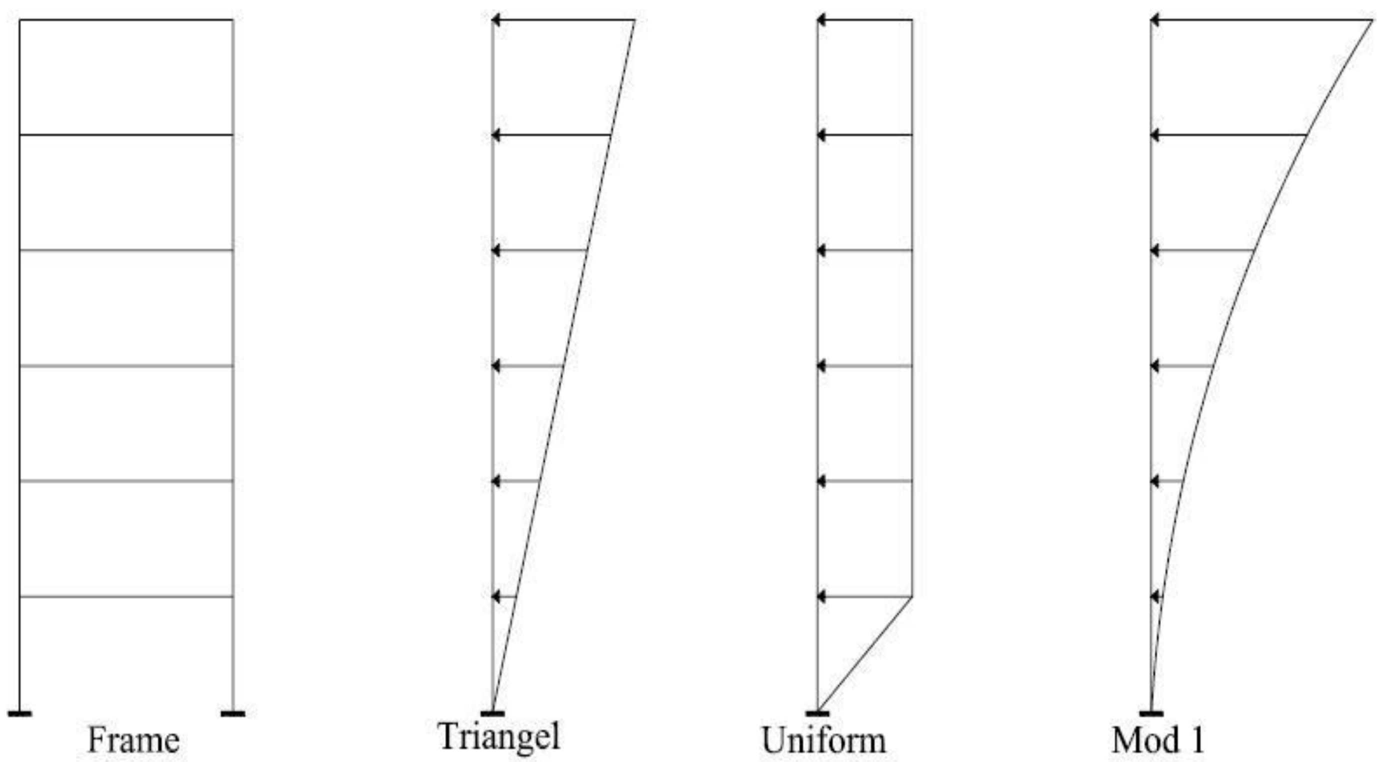

Figure 2. Load pattern of lateral load on pushover analysis.

model of moment beam is applied and due to systematic characteristics of steel frames (having similar bays and heights) the assumptions of the behaviour of the structure conform to the portal model. Therefore, the inflection point of deformation curve versus lateral load occurs in the middle of any connection and it is possible to separate connections from their inflection points and consider a substructure to be studied separately. The simulation of substructure model would behave the same as the real frame by considering a set of suitable boundary conditions. In this case, a subtraction as shown in Figure 3A consisting of a beam with the length of half of its bay in the frame and a column with a height of equally half of its upper and lower columns (the whole of the structure) in the frame which are tightly connected to each other is modelled. In the case of studying the behaviour of compression members and buckling for any type of columns, the one-member behaviour model is considered. To obtain this, the behaviour of axial deflection of members is assumed to be full elasto-plastic in tension. In order to determine the behaviour of compression members for considering the post buckling, it is assumed that the member has an initial curve which is expressed as the initial incompleteness. The quantity of its first deviation from the middle of the opening is the typical value 0.001 L (L is the length of the member). The reaction of axial load versus axial displacement considering variant slenderness of members (KL/R) is calculated by performing nonlinear geometric analysis using finite element method. Afterwards, by the help of piecewise-linear line making method, an ideal relation of axial stress-stain could be attained (Crisfield, 1991). To reach this aim, the nonlinear geometric and material analysis is performed in ANSYS software. This analysis is a sort of nonlinear static overload analysis in which the effects of large deformation are reckoned. The length curve method is used in order to trace the equilibrium path and pass the critical point through the over critical zone (Stafford, 2000). In this part, the axial load-displacement diagram considering different slenderness of elements followed by stress-strain diagram under pressure is obtained using the procedure mentioned in previous part. There are some samples of the behaviour of elements illustrated in Figure 3b in order to consider their curvature.

In order to define the behaviour of structural elements, a comparison between FEMA365 (2000) relations and the finite element model results was accomplished which proved that the moment-rotation relations in joints are in good agreement with the proposed model in FEMA. But

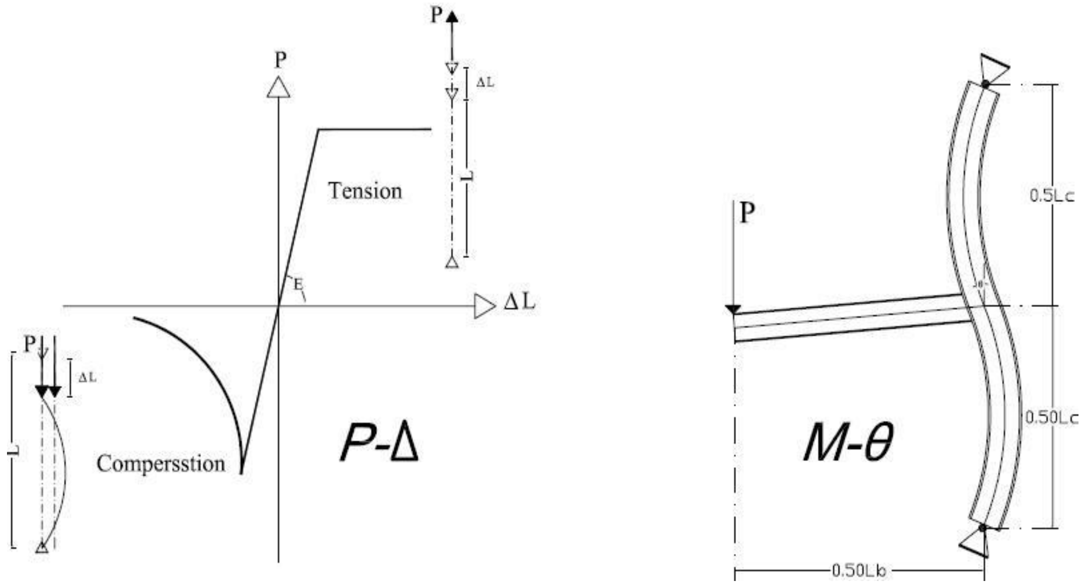

Figure 3. Finite elements modelling of rigid connection and buckling of a single bar.

FEMA relations are overdesigning with respect to columns which have the possibility of buckling. In this research, the FEMA model was used for modelling the behaviour of joints and the results of finite element were utilized for bar model.

NUMERICAL STUDY

In order to study the uncertainties mentioned in the performance point, three groups of buildings with low, middle and high rise steel moment-resistant frames were examined.

Small rise building shown in Figure 4 is a building that its height is shorter than its other dimensions and high rise building is a kind that its height is at least three times longer than its other dimensions (Iranian National Building Code: Part 6, 2006). There is another group of buildings which is placed between these two groups as middle rise building which is a building with medium height (Iranian National Building Code: Part 6, 2006). These frames could bear the average dead load of 650 kg/m^2 on the stories and 600 kg/m^2 on the roof; moreover, the mean of live load of stories and the roof are considered 200 and 150 kg/m^2, respectively.

The loading of stories and the roof are changed in order to change the mass of the structure. The geometric characteristics and other information about frames are summarized in Table 4.

These frames are designed and loaded based on Iranian earthquake code 2800 (1999), Iranian National Building Code: Part 6 (2006) and Iranian National Building Code: Part 10 (2006). The configuration of these frames are considered 4-m bay with 3-m height as common dimensions. The distance between frames is considered 4 m. With respect to the typical classification in the design of structural elements, the change in cross-section area is considered in two or three stories for the columns and one beam has been used in each story. Assuming that the frames being studied belong to a symmetric structure, columns are chosen to be box section and shape sections for beams.

SENSITIVITY ANALYSIS AND THE RESULT ASSESSMENT

The sensitivity analysis is one of the important and essential steps in designing structures. The sensitivity of a structural response is its variation per a unit change in designing variables such as dimensions or the strength of elements. Nonlinear procedure is typically used for structural analysis in performance based design. Therefore, the nonlinear sensitivity analysis of the structure is required. There are two general approach for performing sensitivity analysis including finite-difference and analytical method. Despite the simplicity of finite-difference method, its results might not be reliable for nonlinear sensitivity analysis (Habibi et al., 2007). Therefore, in this research, performance point is defined with two methods introduced by ATC40 (1997) and FEMA356 (2000) with respect to the uncertainty

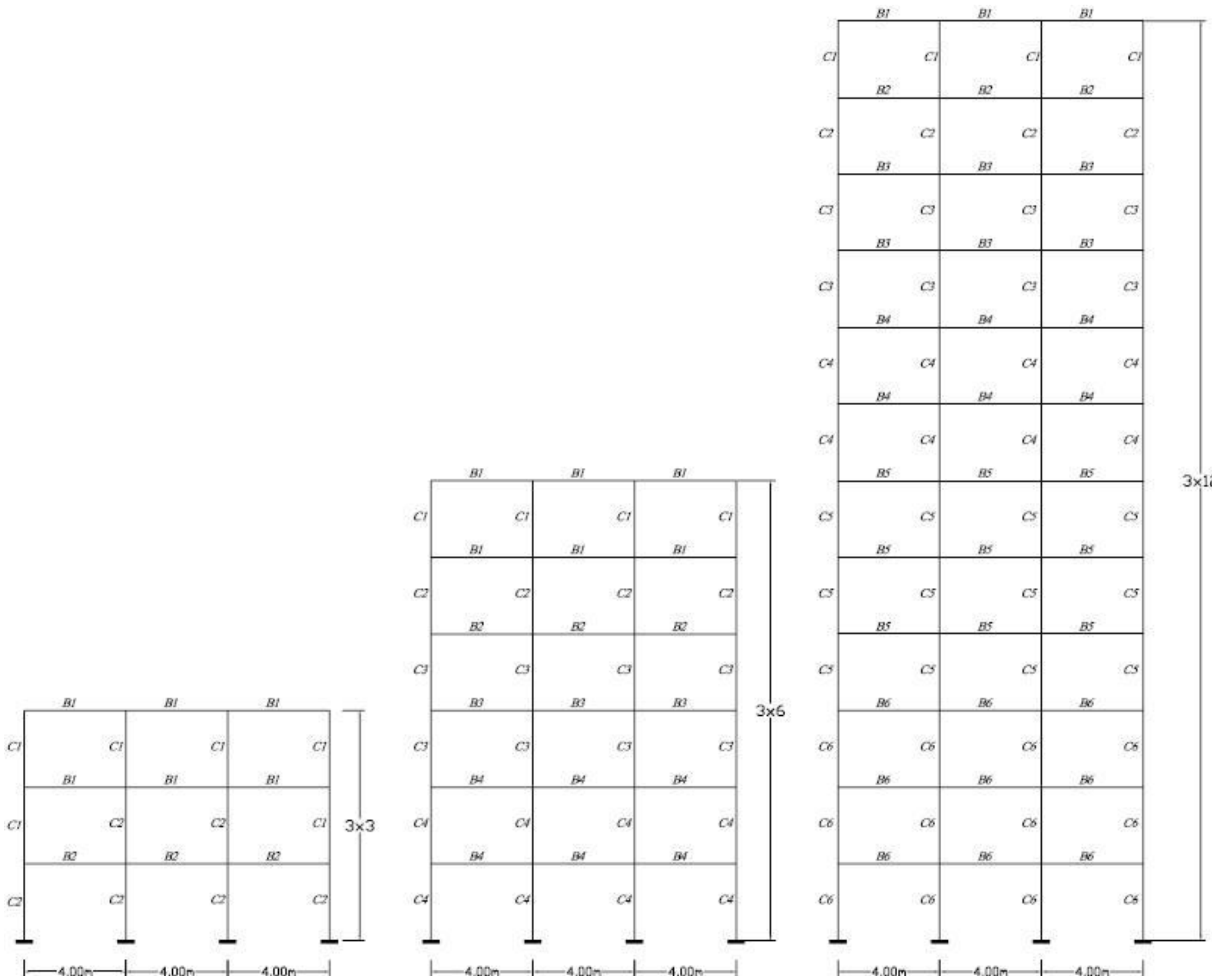

Figure 4. Three types of frames.

Table 4. Random variables statistical characteristics.

C1	BOX 20x20x1.0
C2	BOX 18X1810.8
C3	BOX 30X30X1.2
C4	BOX26X26X1.0
C5	BOX 38X38X1.5
C6	BOX 34X34X1.2
B1	IPE27
B2	IPE30
B3	IPE33
B4	IPE36
B5	IPE40
B6	IPE45

Mean values: Es = 21000000 Kg/cm^2; Fy = 2400 Kg/cm^2; Fu = 3700 Kg/cm^2.

Table 5. Percent of variation and uncertainty parameters for three-story frame.

Variable	Method	Lateral load pattern		
		Mode 1	Triangular	Uniform
DL	ATC40	5.362	5.149	6.34
	FEMA356	4.683	4.641	5.333
LL	ATC40	1.244	2.062	3.707
	FEMA356	0.939	1.032	1.061
Fy	ATC40	2.618	1.432	4.862
	FEMA356	2.167	1.123	6.715
Es	ATC40	3.173	2.953	2.492
	FEMA356	3.254	3.164	2.987
Area	ATC40	19.147	18.925	17.223
	FEMA356	15.698	15.517	15.723

Table 6. Percent of variation and uncertainty parameters for six-story frame.

VariableMethod		Lateral load pattern		
		Mode 1	Triangular	Uniform
DL	ATC40	6.361	5.897	5.478
	FEMA356	5.754	5.679	5.402
LL	ATC40	2.743	1.617	2.428
	FEMA356	1.235	1.177	1.191
Fy	ATC40	1.176	1.013	1.271
	FEMA356	0.358	0.455	0.823
Es	ATC40	3.034	3.209	3.319
	FEMA356	3.183	3.342	3.356
Area	ATC40	11.682	11.362	11.412
	FEMA356	10.612	10.612	10.311

parameters in each frame. The performance point is determined probabilistically using mentioned methods based on three lateral load patterns. Afterwards an appropriate regression is obtained on results for each variable with uncertainty and the mean of uncertainty parameters and coefficient of variation of the uncertainty parameters is studied. The results of this research are shown in Tables 5 to7.

According to Tables 5 to 7, it is obvious that generally, the performance point has the most sensitivity to the

Table 7. Percent of variation and uncertainty parameters for twelve-story frame.

Variable	Method	Lateral load pattern		
		Mode 1	Triangular	Uniform
DL	ATC40	5.873	5.577	5.378
	FEMA356	5.178	5.142	5.114
LL	ATC40	1.245	1.062	1.108
	FEMA356	1.082	1.074	1.069
Fy	ATC40	1.231	2.926	1.021
	FEMA356	0.625	0.714	0.868
Es	ATC40	2.968	3.094	5.512
	FEMA356	3.225	3.255	3.259
Area	ATC40	10.012	9.886	9.307
	FEMA356	8.898	8.952	8.864

geometric characteristics of cross sections. Therefore, these parameters hold the most uncertainty in the process. The dead load intensity is in the second place and live load is of more uncertain behaviour compared to dead load as mentioned before where as uncertainty of the dead load is more than the live load in determining the performance point. This is because of more percentage of cooperation of dead load in contrast to the live load in designing process. As expected, finding the performance point by the ATC40 method has more uncertainty compared with the FEMA356 method. Only by changing Yung's modulus the FEMA356 method becomes more sensitive compared to ATC40. In small rise buildings, the inverted triangle lateral load pattern has the least uncertainty and the load pattern based on the uniform distribution has the most uncertainty. This case is justifiable regarding the behaviour of small rise buildings (the behaviour mode in these buildings is on basis of shear). In buildings with medium height, the uniform distribution pattern has the least and the lateral load based on the first mode of vibration has the most uncertainty. Participation of higher modes in this group of structures confirms supports to this fact. On the other hand, in middle rise buildings with a mixed behaviour of shear-moment behaviour, the distribution of uniform lateral load is more logical. In high rise buildings, the uniform load pattern has the least uncertainty and still the load pattern has the most uncertainty on the basis of the first mode of vibration. In this group of structures also because of behaviour in the form of pendulum and an upper mode affecting structural behaviour, the results obtained are confirmed. The least sensitivity of the

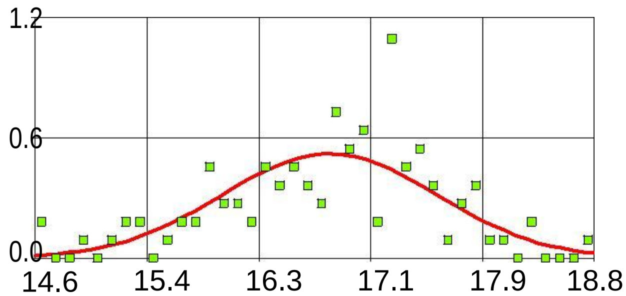

Figure 5. Performance point variation with respect to the dead load for three-story frame.

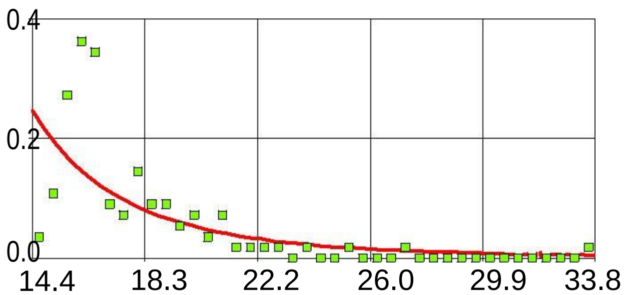

Figure 6. Performance point variation with respect to the geometry for three-story frame.

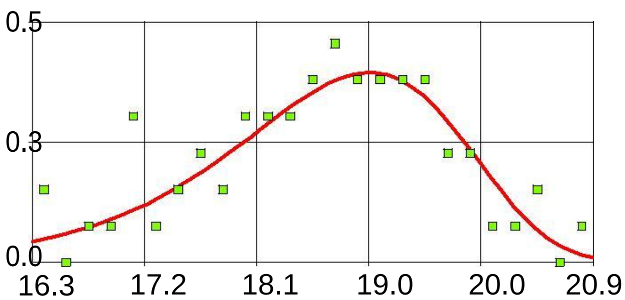

Figure 7. Performance point variation with respect to the dead load for six-story frame.

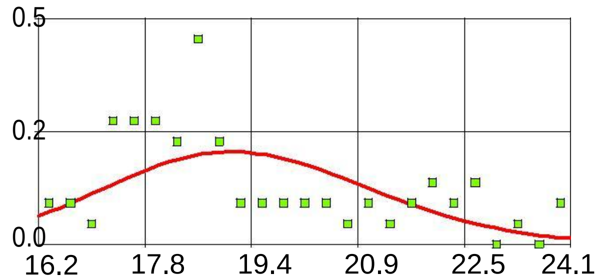

Figure 8. Performance point variation with respect to the geometry for six-story frame.

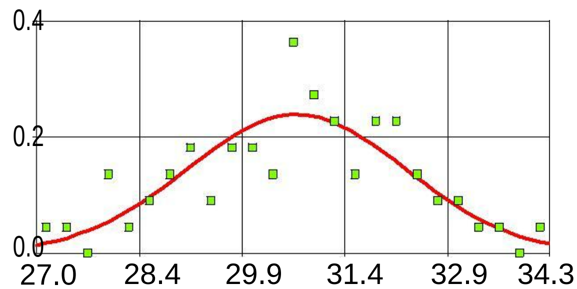

Figure 9. Performance point variation with respect to the dead load for twelve-story frame.

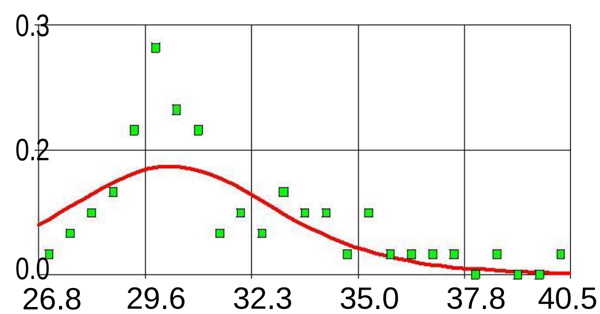

Figure 10. Performance point variation with respect to the geometry for twelve -story frame.

performance point is related to the tension of steel elements. The reason of this fact could be explained in a way that the conditions must be suitable enough for yielding of the structure. The factors like partial and local buckling, the connection type, arrangement of mecha-nisms, etc (Using the whole capacity of the sectional area) are dominant factors in the problem. This matter would get worse by increasing the height of structures. The rest of the required results and comments are shown in Tables 5 to 7. In order to consider the statistical results of variation, the performance point relating to the sensitive parameters (geometric characteristics and dead load) are shown on the basis of lateral load pattern and the method of defining the performance point presenting the least amount of coefficient of variation (these two groups are highlighted by yellow and green colours in Tables 5 to 7). The curves of variation of the performance point for each group of building are also shown. Furthermore, the most suitable probability function with parameters relating to data is attained for considering the best probability function corresponding to the variation of statistical data, using a variety of functions (Figures 5 to10).

CONCLUSION

In this research, the study of the uncertainties in performance based design method was considered. The uncertainty factors in this paper were classified into three groups: Uncertainties in resistances, loads and the analyzing methods. The uncertainty related to the strength of structures has a direct relation with the geometrical characteristics of cross sections and the quality of materials. Results show that the geometric characteristics are the principal sources of uncertainty. Therefore this parameter must be considered as an uncertainty parameter. The parameters relating to the characteristics of materials have the least uncertainty and could be assumed as explicit parameters. This subject could be reasonable because of the production of steel sections in the factory. The second resources of uncertainty are the loads with considerable uncertainties. Therefore, the probabilistic study of the structures considering these parameters especially dead loads is essential. The ambiguities which are obtainable in modelling the components of the behaviour of structures and also the limited information about earthquake characteristics, the characteristics of the function acceleration and interaction between soil and structure increase the usage of over loading probabilistic analysis. Therefore, for any groups of buildings with respect to their height, considering appropriate lateral load pattern which was explained in this paper and using precise modelling of the behaviour of structural components, uncertainties could be minimized in the earthquakes.

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