현행 내진 설계규준(UBC)으로 설계된 구조물의 내진 신뢰성 평가

Evaluation of Seismic Reliability of Structures Designed According to Current Seismic Design Provision(UBC)

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요 약

이 논문의 목적은 현행법규(Uniform Building Code, NEHRP 설계규준등)에 따라 설계된 구조들의 성능 및 안정성을 평가하는데 있다. 구조물의 내진성능은 신뢰성 (Reliability)으로 표현할 수 있다. 구조물의 그의 수명동안 내진에 대한 신뢰성 해석을 하기 위해서는 주어진 구조물들의 많은양의 동적반응 해석을 요구하는 것이 일반적이다. 따라서 구조물의 신뢰성 해석을 위하여 구조물이 위치한 지역에 많은양의 지진기록들이 요구된다. 이 논문에서는 인위적인 지진들(artificial earthquakes)을 부정착 임계 파정법(nonstationary randon process)을 이용해 만들었고 구조물은 Uniform Building Code와 AISC 허용응력 설계지침서 (AISC aloowables stress design manual)에 따라서 설계하였다. 이 논문에서는 주어진 지진하중에 대한 구조물의 반응(response)은 구조물의 비선형 반응해석과 반응변위수정계수들을 이용한 간략화된 동위 구조물 해석(ENS)으로 얻었다. 이 논문에서는 구조물의 내진 성능을 평가하였다. 또한 동위구조물(Equivalent Nonlinear System (ENS))을 이용한 해석과 구조물의 비선형 반응해석의 결과들을 비교하여 동위구조물을 이용한 방법의 정확성도 평가하였다.

Abstract

The purpose of this study is to evaluate performance and safety of structures designed according to current seismic code or provisions (e.g., Uniform Building Code(UBC), NEHRP provisions, etc.) during lifetime of structures. The performance is represented in terns of reliability in this paper. To perform reliability analyses, a large number of time history response analyses for a given structure are usually required. In this study, to perform reliability analyses ground motions are generated based on nonstationay random process and structures are designed based on UBC. In this paper, responses of structures under a given earthquake is evaluated using dynamic nonlinear time history analyses and also an equivalent nonlinear system (ENS) with response scaling factors. The ENS system is described in the companion paper. Therefore, this paper evaluates the seismic performance of structures and also verify the accuracy of ENS.

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이 논문에 대한 토론을 1996년 3월 31일까지 본 학회에 보내주시면 1996년 9월호에 그 결과를 게재하겠습니다.

1. INTRODUCTION

Structural Reliability under earthquakes has always been a major concern to structural designers. It is not easy problem to evaluate the seismic reliability since there are large uncertainties involved in seismic loadings. These uncertainties are considered in current codes in the form of load factors. These load factors have been often determined based on engineering judgement, expertise and social-economic considerations. In current seismic code or provisions(UBC, NEHRP, ATC, Seismic provisions in Korea etc.), an equivalent lateral force procedure is used which takes into account the seismicity of the region where the building is located, a design spectrum, and the effect of ductility of structureusing a reponse modification factor. A linear static procedure is then performed to the required strength and stiffness of the building components. However, the safety implied in the code procedure is not given even if above mentioned assumptions and subjective manners of code committee members are involved in the code procedures.

Many researchers have evaluated the seismic reliability of design code procedures(Shinozuka et al 1989, Hwang et al 1991, and Wen et al 1991). A number of nonlinear dynamic response analyses needs to be performed to evaluate the reliability of a structure in their studies, whcih requires excessive computational efforts. Specially this computational problem will be more serious when one atempts to calibrate the design parameters in codes based on reliability(Han, S. W., 1994), since many structures and many locations need to be considered.

As mentioned in the companion paper, Cornell et al have developed an equivalent linear SDOF system(ELSS) which contains the 1st mode of a structure under consideration to evaluate reliabilities of structures. However, the reliabilities obtained using ELSS is not quite comparable to the reliabilities using nonlinear dynamic response analyses (Example will be shown in this paper). In this study reliabilities of structures are evaluated based on the nonlinear equivalent system (ENS) which is introduced in the companion paper. Specially moment resisting steel frames (SMRSF) are considered. To perform reliability analyses, artificial earthquakes are generated by nonstationary random process which is used by Yeh and Wen (1989). Reliabilities of structures during their lifetimes are evaluated based on the responses from nonlinear dynamic response analysis, ELSS and ENS. Based on the calculated reliabilities of structures, the performance of a structure will be discussed and the results from equivalent system will be compared.

2. SEISMIC RISK ANALYSIS USING EQUIVALENT SYSTEMS

The seismic performance of a structure can be measured in terms of a limit state probability. Limit states represent the various states of undesirable behavior of structures such as yielding, excessive deflectin, instability, buckling, severe damage, etc. In this study, the limit states are defined in terms of interstory or total drift. It is reasonable to define the limit state using a frift limit, since SMRSFs designed according to current seismic codes are usually governed by lateral stiffness requirements. In the Uniform Building Code (UBC−1991), the maximum interstory frift calculated (3Rw/8) δ_x must be less than 1.5 percent of the story height. (δ_x is the elastic drift computed by the equivalent lateral force method in UBC) The elastic drift limit is also limited to $0.04/R_w$ or 0.005 times the story height, whichever is smaller. The global limit state probability can be represented as follows:

$$P_G = P(U_G > x_{U_G}) \quad (1)$$

where x_{U_G} represents displacement threshold at the top floor. In Eq. (1), a global displacement threshold, x_{U_G}, can be expressed as the value of the global ductility threshold(x_{μ_G})multiplied by global yield displacement (U_y). Thus, Eq. (1) can be written as follows:

$$= P(\frac{U_G}{U_y} > x_{\mu_G}) \quad (2)$$

$$= P(\mu_G > x_{\mu_G}) \quad (3)$$

The local limit state probability can be represented as follows:

$$P_L = P(\max_i(U_{L_i}) > x_{U_L}) \quad (4)$$

$$= P(\max_i(\frac{U_{Li}}{U_{yi}}) > x_{\mu_L}) \quad (5)$$

$$= P(\max_i(\mu_{L_i}) > x_{\mu_L}) \quad (6)$$

where P_G is the global limit state probability and P_L is the local limit state probability, x_{U_L} is the interstory drift threshold and x_{μ_L} is a local (interstory) ductility threshold. U_G is the maximum displacement at the top floor, and U_{Li} is the maximum interstory drift due to a given earthquake obtained by a nonlinear dynamic analysis of a MDOF system. To avoid extensive computation, the limit state probabilities defined by Eqs.(1) and (2) are calculated using either the Equivalent Linear System(ELSS) with the scaling factors F and C or the Equivalent Nonlinear System(ENS) with a global response scaling factor (R_G) and a local response scaling factor (R_L) as described below.

3. SEISMIC RISK ANALYSIS USING EQUIVALENT LINEAR SDOF SYSTEM

The procedures using the ELSS developed by Cornell and others (See companion paper) for evaluating the seismic risk associated with a structure are as follows:

$$P_G = P(\mu_G > x_{\mu_G}) \approx P(\frac{S_a}{E(C)E(F_{\mu_G})} > S_{a_{yld}}) \quad (7)$$

$$P_L = P(\max_i(\mu_{Li}) > x_{\mu_L}) \approx P(\frac{S_a}{E(C)E(F_{\mu_L})} > S_{a_{yld}}) \quad (8)$$

In these equations, S_a is the linear spectral acceleration of the first mode of a MDOF structure. S_{ayld} is the yield level spectral acceleration of the MDOF structure. E(.) denotes the mean value.

The factor F_{μ_G} is the global nonlinear spectral reduction factor which is defined for a given global ductility (μ_G) as the scaling factor for ground motion necessary to achieve a displacement at the top of the structure which is μ_G times larger than the displacement experienced by the top of the structure when incipient yield occurs anywhere in the structure. Similarly, F_{μ_L} is the local nonlinear spectral reduction factor which is defined for a given local ductility threshold (μ_L) as the scaling factor for ground motion necessary to achieve the local μ_{L_i} in at least one element of the structure (Bazzurro and Cornell, 1992). Hence, the spectral acceleration at a certain damage level (e.g., $\mu_G=4$), $S_a^{DM(\mu_G=4)}$, can be evaluated as follows:

$$S_a^{DM(\mu_G=4)} = \frac{S_a}{F_{\mu_G=4}} \quad (9)$$

The MODF response factor (C) in Eq.(7) and (8) can be evaluated using following equations:

$$C = \frac{DI_{sdof}}{DI_{mdof}} \tag{10}$$

$$DI_{mdof} = \max_i\left(\frac{|U_i|_{max}}{U_{i,c}}\right) \tag{11}$$

$$DI_{sdof} = \frac{|U_1|_{max}}{U_{1,c}} \tag{12}$$

where $|U_i|_{max}$ is the maximum value of the i th story drift from linear analysis, and $U_{i,c}$ is the linear story drift capacity. In Eq. (11) and (12), the maximum story drift ($|U_i|_{max}$) can be easily calculated by a response spectrum analysis as follows(Chopra, 1975):

$$|U_i|_{max} = \sum_{i=1}^{N}(\phi_{i,n} - \phi_{i-1,n})\frac{L_n}{M_n}S_{dn} \tag{13}$$

where $\phi_{i,n}$ is the normalized displacement of n th mdoe shape of i th floor, m_i is the lumped mass at the i th floor, and S_{dn} is the spectral displacement of the n th mode.

In order to test the accuracy of ELSS, the seismic risks associated with a 2 story SMRSF evaluated using the ELSS are also compared with the risks using nonlinear MDOF system.

4. SEISMIC RISK ANALYSIS USING EQUIVALENT NONLINEAR SYSTEM

The equation for evaluating the global limit state probability using the ENS proposed in this study is as follows:

$$P_G \approx P\left(\frac{U_E}{R_G} > x_{U_G}\right) \tag{16}$$

For evaluating the local limit state probabilities using the ENS, Eq.(4) is divided by U_{yi} as follows:

$$P_L = P\left(\max_i(U_{Li}/U_{yi}) > \frac{x_{U_L}}{U_{yi}}\right) \tag{17}$$

Using Eq.(4),

$$\approx P\left(\frac{U_G/U_y}{R_L} > \frac{x_{U_L}}{U_{yi}}\right) \tag{18}$$

Since $R_G = U_E/U_G$(companion paper),

$$= P\left(\frac{U_E/U_y}{R_G R_L} > \frac{x_{U_L}}{U_{yi}}\right) \tag{19}$$

Then, multiply both sides by U_y,

$$= P\left(\frac{U_E}{R_G R_L} > x_{U_L}\left(\frac{U_y}{U_{yi}}\right)\right) \tag{20}$$

Since,

$$x_{U_G} \approx x_{U_L}\cdot\left(\frac{U_y}{U_{yi}}\right) \tag{21}$$

the equation for evaluating the local limit state probabilities can be approximated as:

$$P_L \approx P\left(\frac{U_E}{R_G R_L} > x_{U_G}\right) \tag{22}$$

In the above equations, U_E is the maximum displacement of the ENS. R_G is the global response scaling factor to account for the difference between the maximum displacement of the equivalent system(ENS) and that of the MDOF structure. Similarly, R_L is the local response scaling factor applied to the global response of th MDOF structure to obtain the maximum local (interstory) drift response.

5. EVALUATION OF LIMIT STATE PROBABILITIES AND VERIFICATION OF THE PROPOSED METHOD

Limit state probabilities of the representative structures located at either santa Monica Boulevard site in L.A. or at the Imperial Valley site are evaluated for a time window of 1995−2045 by the proposed method and compared with the results of analyses of MDOF systems (NMS). The Santa Monica Boulevard site in Los Angeles is 60 Km from the Imperial fault. For evaluating the limit state probabilities of a sturcutre at each site, ground motions are simulated. The Method of simulation is outlined in the following.

The potential earthquakes that present a threat to these sites can be modeled as either characteristic earthquakes or non-characteristic earthquakes. Characteristic earthquakes represent the earthquakes which occur along a fault. These earthquakes have a relatively regular recurrence period and a narrow range of magnitudes. The probability of occurrence of a characteristic earthquake depend on the elapsed time since the last earthquake. Thus timd dependence is typically modelled as a renewal process with lognomally distributed recurrence time. Non-characteristic earthquakes are minor local events which are randomly distributed in time (i.e., occurrences are independent of the time since the last earthquake). Hence, the occurrence of non-characteristic earthquakes is modeled as a Poission process. Besides the recurrence time, the major parameters of the characteristic earthquakes for seismic risk analysis are magnitude (M), epicentral distance to the site (R), and attenuation function. For non-characteristic earthquakes, the major parameters are local intensity (MMI) and duration (t_d). Using these parameters, future earthquakes in these sites can be simulated. The parameters of ground motion model used for the two sites are presented in the study by Eliopoulos and Wen(1991). The synthetic accelograms are modeled as a sample function of a non stationary random process exhibiting frequency modulation function, intensity function and power spectral density function. Details of the ground motion model are described in the study by Yeh and Wen (1989).

For the Imperial Valley site, two sets of ground motions are generated. One set is generated using parameters determined from the differential array seismic station records obtained during the 1979 Imperial Valley Earthquake. This set contains the effect of the fault repture propagating toward the site. The other set is generated based on parameters obtained fromthe 1940 El Centro earthquake record. This latter set has the opposite directivity. For the L.A. site, seismic hazard due to both characteristic earthquakes and non-charactreristic earthquakes is investigated: hence, two sets of ground motions are generated for this site. Since each site has two different sources of ground motions, the contributions to the overall lifetime limit state probability from these two different sources are treated separately. Hence, Eqs. 16 and 17 need to be modified. Global limit state probability can be calculated as follows:

$$P_G = 1 - (1-P_{g1})(1-P_{g2}) \qquad (23)$$

For La site:

$$P_{g1} = P(U_G > x_{U_G}|Ch)P(Ch) \qquad (24)$$

$$P_{g2} = P(U_G > x_{U_G}|Nch)P(Nch) \qquad (25)$$

For Imperial Valley site

$$P_1 = P(U_G > x_{U_G} | Ch_t)P(Ch_t) \qquad (26)$$

$$P_2 = P(U_G > x_{U_G} | Ch_a)P(Ch_a) \qquad (27)$$

where U_G is the maximum displacement at the top of the structures and x_{U_G}. This displacement is evaluated using response analysis of the NMS or analysis of the ENS using the factors R_G and R_L. The terms Ch, Nch, Ch_t and Ch_a represent the occurrence of the characteristic earthquake, non-characteristic earthquake, characteristic earthquake toward the Imperial Valley site, and characteristic earthquake sway from this site, respectively. Local limit state probability can be calculated as follows:

$$P_L = 1 - (1-P_{l1})(1-P_{l2}) \qquad (28)$$

For LA site:

$$P_{l1} = P(\max_i(U_{Li} > x_{U_L} | Ch)P(Ch) \qquad (29)$$

$$P_{l2} = P(\max_i(U_{Li} > x_{U_L} | Nch)P(Nch) \qquad (30)$$

For Imperial Valley site:

$$P_{l1} = P(\max_i(U_{Li} > x_{U_L} | Ch_t)P(Ch_t) \qquad (31)$$

$$P_{l2} = P(\max_i(U_{Li} > x_{U_L} | Ch_a)P(Ch_a) \qquad (32)$$

where i is the story number in a structure. U_{Li} can be evaluated using the analysis of a NMS or an ENS with the factors R_L and R_G. x_{U_L} is story drift threshold. Additional detail of the above equations are given by Eliopoulos and Wen(1991).

6. CONCLUSIONS

The limit state probabilities of the structures located at both the L.A. site and the Imperial Valley site are evaluated based on the response from analyses of NMS, ELSS with the factors F and C, and ENS with the Factors R_G and R_L. T perform thee analyses, 50 non-characteristic and 20 characteristic earthquakes are generated for the L.A. site, and 20 characteristic earthquakes away from the site and 50 characteristic earthquake toward the site are generated for the Imperial Valley site. Hence, there are 70 maximum responses for each structure at each site. This study uses the same representative structures of the companion paper. Based on these maximum responses, the limit state probabilities are eval-

Table 1. Global limit state probability(L.A. site)

Structure No.	Used System	0.5%of height	1.0%of height	1.5%of height	2.0%of height	2.5%of height	3.0%of height
1	NMS	0.2415	0.0749	0.0387	0.0241	0.0167	0.0123
1	ENSS	0.2443	0.0741	0.0381	0.0237	0.0163	0.0120
2	NMS	0.3021	0.0906	0.0476	0.0300	0.0210	0.0156
2	ENSS	0.3050	0.0898	0.0468	0.0294	0.0205	0.0152
2	ELSS	0.3165	0.0804	0.0429	0.0259	0.0178	0.0140
3	ENSS	0.2459	0.0708	0.0368	0.0231	0.0161	0.0120
4	NMS	0.2477	0.0760	0.0403	0.0256	0.0180	0.0135
4	ENSS	0.2356	0.0692	0.0361	0.0227	0.0158	0.0112
5	NMS	0.2334	0.0695	0.0364	0.0230	0.0161	0.0120
5	ENSS	0.2513	0.0748	0.0392	0.0247	0.0172	0.0128
6	NMS	0.1650	0.0475	0.0233	0.0140	0.0094	0.0068
6	ENSS	0.1495	0.0422	0.0202	0.0119	0.0079	0.0056
7	NMS	0.1286	0.0375	0.0182	0.0108	0.0072	0.0052
7	ENSS	0.1212	0.0340	0.0161	0.0094	0.0062	0.0044

Table 2. Local limit state probability(L.A. site)

Structure No.	Used System	0.5%of height	1.0%of height	1.5%of height	2.0%of height	2.5%of height	3.0%of height
2	NMS	0.3588	0.1153	0.0621	0.0399	0.0282	0.0212
2	ENSS	0.4229	0.1235	0.0637	0.0401	0.0280	0.0208
2	ELSS	0.3658	0.0869	0.0515	0.0303	0.0243	0.0165
3	NMS	0.4282	0.1345	0.0717	0.0460	0.0324	0.0243
3	ENSS	0.4367	0.1278	0.0639	0.0402	0.0281	0.0209
4	NMS	0.4676	0.1536	0.0831	0.0538	0.0382	0.0289
4	ENSS	0.4335	0.1268	0.0641	0.0404	0.0282	0.0210
5	NMS	0.3986	0.1224	0.0646	0.0412	0.0289	0.0216
5	ENSS	0.4615	0.1384	0.0703	0.0444	0.0311	0.0232
6	NMS	0.2990	0.0917	0.0468	0.0289	0.0198	0.0145
6	ENSS	0.3403	0.0917	0.0441	0.0262	0.0174	0.0125
7	NMS	0.2401	0.0748	0.0377	0.0230	0.0156	0.0113
7	ENSS	0.2914	0.0789	0.0377	0.0222	0.0147	0.0105

Table 3. Global limit state probability(Imperial valley site)

Structure No.	Used System	0.5%of height	1.0%of height	1.5%of height	2.0%of height	2.5%of height	3.0%of height
1	NMS	0.8264	0.7045	0.2956	0.1155	0.0383	0.0109
1	ENSS	0.8264	0.6902	0.2879	0.0916	0.0245	0.0062
2	NMS	0.8258	0.5519	0.1930	0.0589	0.0166	0.0045
2	ENSS	0.8230	0.5595	0.2154	0.0568	0.0140	0.0051
2	ELSS	0.8218	0.4835	0.1652	0.0421	0.0110	0.0029
3	NMS	0.8253	0.4241	0.0886	0.0147	0.0023	0.0003
3	ENSS	0.7942	0.4551	0.1020	0.0154	0.0021	0.0003
4	NMS	0.8252	0.4607	0.1209	0.0255	0.0051	0.0001
4	ENSS	0.7927	0.4495	0.1023	0.1023	0.0160	0.0083
5	NMS	0.8247	0.4282	0.0923	0.0157	0.0026	0.0004
5	ENSS	0.8026	0.4859	0.1192	0.0193	0.0028	0.0004
6	NMS	0.8178	0.1820	0.0512	0.0012	0.0001	0.7e-4
6	ENSS	0.7676	0.1506	0.0111	0.0008	0.0001	0.4e-4
7	NMS	0.7676	0.0821	0.0054	0.0004	0.3e-4	0.2e-5
7	ENSS	0.6702	0.0824	0.0065	0.0005	0.4e-4	0.3e-5

Table 4. Local limit state probability(Imperial valley site)

Structure No.	Used System	0.5%of height	1.0%of height	1.5%of height	2.0%of height	2.5%of height	3.0%of height
2	NMS	0.8245	0.6632	0.3522	0.1718	0.0792	0.0350
2	ENSS	0.8257	0.7000	0.3798	0.1761	0.0759	0.0313
2	ELSS	0.8256	0.6083	0.3298	0.1009	0.0500	0.0179
3	NMS	0.8264	0.8237	0.4890	0.1695	0.0494	0.0133
3	ENSS	0.8213	0.6951	0.3931	0.1352	0.0367	0.0093
4	NMS	0.8264	0.8264	0.6529	0.3364	0.1428	0.0549
4	ENSS	0.8212	0.6987	0.4034	0.1453	0.0415	0.0109
5	NMS	0.8264	0.8178	0.4769	0.1634	0.0441	0.0110
5	ENSS	0.8237	0.7235	0.4480	0.1738	0.0520	0.0142
6	NMS	0.8264	0.6832	0.2281	0.0416	0.0065	0.0010
6	ENSS	0.8254	0.5865	0.1688	0.0321	0.0056	0.0010
7	NMS	0.8264	0.5596	0.1381	0.0236	0.0038	0.0006
7	ENSS	0.8222	0.4473	0.1081	0.0219	0.0044	0.0009

uated by Eq.(23) and Eq. (28). Tables 1 and 2 show the global and local limit state probabilities of the structures for the L.A. site using NMS and ENS. Tables 3 and 4 also show the global and local limit state probabilities of the structures for the Imperial Valley site. Six different global limit states and six different local limit states are considered. The global limit states for the dislacement at the top of the structure are set at 0.5%, 1.0%, 2.0%, 2.5% and 3.0% of the height of the structure. Similarly, the local limit states for interstory drift are set at 0.5%, 1.0%, 1.5%, 2.0%, 2.5% and 3.9% of the story height. In Figures 3.30 and 3.31, limit state probabilities for the L.A. site obtained using NMS and ENS are are compared for global and local limit states. Similarly,

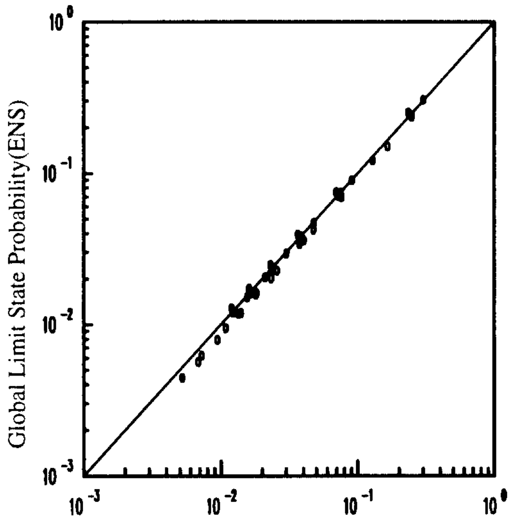

Figure 1. Comparison global limit state probabilities of ENS and NMS(L.A. site)

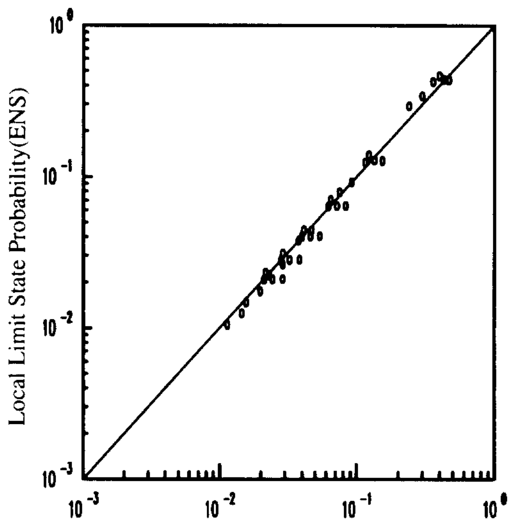

Figure 2. Comparison of local limit state probabilities of ENS and NMS(L.A. site)

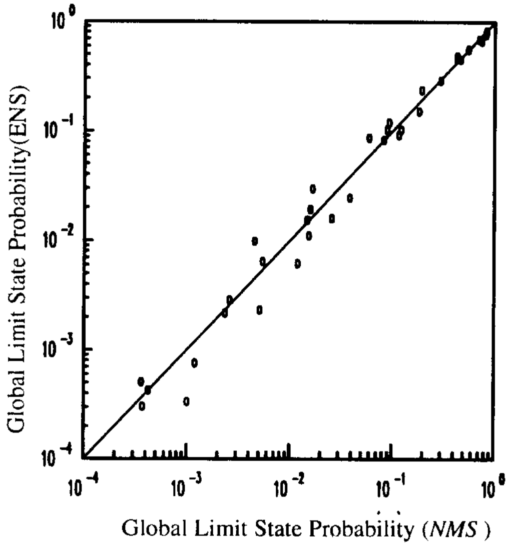

Figure 3. Comparison of global limit state probabilities of ENS and NMS(Imperial Valley site)

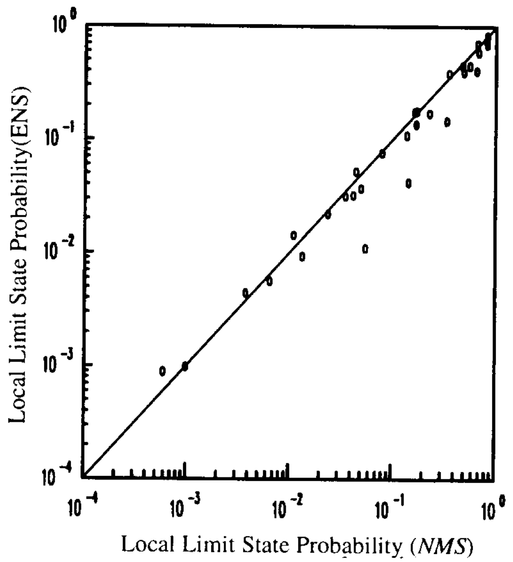

Figure 4. Comparison of local limit state probabilities of ENS and NMS(Imperial Valley site)

Figures 1 and 2 show the comparisons for the Imperial Valley site. Similarly, Figures 3 to 4 show the deviation of limit state probabilities of ENS from NMS for the Imperial Valley site. The discrepancy between the two sets of limit state probabilities from ENS and NMS for the Imperial Valley site is lager than that for the L.A. site. This result may be attributed to large inelastic behavior of the strucutre at the Imperial Valley site, since the randomness inR_G and R_L increase with the inelastic deformation fo the structure. The results are also shown in Tables 1 to 4. In the view of the generally good comparison, it is concluded that the limit state probabilities based on ENS with R_G and R_L can be used as an approximation to those of nonlinear multi-degree of freedom systems.

The global and local limit state probabilities of a 2 story SMRSF using ELSS with the factors F and C are also calculated and compared to those using NMS. Using the equation presented by Inoue and Cornell(1991), the C factor for the 2 story SMRSF is 0.9. Figure 5 shows the histogram of the deviation of the limit state probabilities of ELSS from the NMS. This figure also show that the limit state probabilites obtained using ELSS is biased. The corresponding deviation of the ENS system limit state probability from those of the NMS is

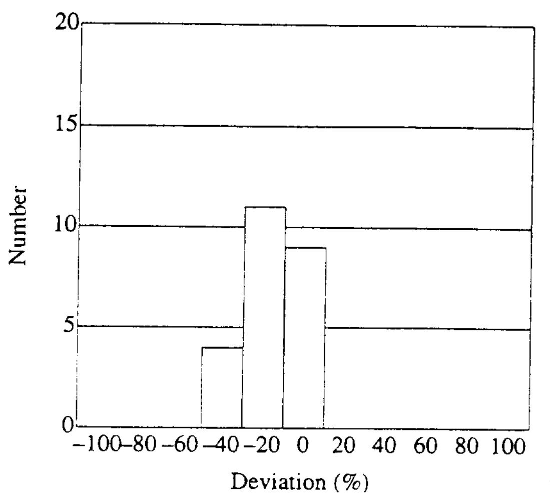

Figure 5. Histogram of differenc of local limit state probabilities (ELSS and NMS) (2 story SMRSF)

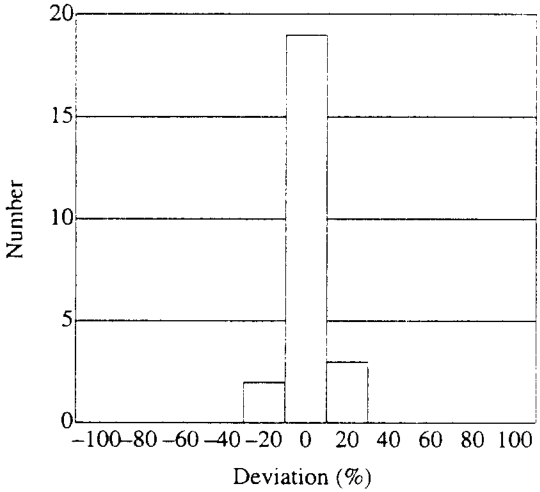

Figure 6. Histogram of difference of local limit state probabilities (ENS and NMS) (2 story SMRSF)

shown in Figure 6. It is seen that accuracy of the ENS method is satisfactory.

From tables 1−4, limit state probabilities for high-rise structures (> 7 stories, ATC-13) which are designed according to seismic design probisions are lower than those for low-rise(1 to 3 stories) and medium-rise structures(4−7 stories). This implies that the design procedures in the current seismic design provisions are more conservative when applied to high-rise structures. This observation is reasonable since the failure of high-rise structure can lead to more serious consequences than that of a low-rise or medium-rise structure. The worst case of limit state probabilities for serviceability and ultimate limit states of L.A. site is 0.4676 and 0.0831 for 50 years(5 story structure), which correspond to annual risks of 0.0093 and 0.0017. These risks are in agreement with suggested acceptable risk level associated with building designed according to building code (Hays, 1985). The worst case of limit state probabilities for Imperial Valley site are 0.8264 and 0.6951, which correspond to annual risks of 0.01652 and 0.0139. These risk levels are not acceptable for building(Hays, 1985). Both sites are classified as th strongest seismic zone in the current seismic provisions. However, the risk level is quite different. This imply that micro-zoning process is required for the seismic design provisions.

It is noted that only regular SMRSF are considered in this study. The methodology may be extended to other structural systems, such as Ordinary Moment Resisting Frame (OMRF), Concentric Braced Frame(CBF), and Eccentric Braced Fame(EBF), etc. Also, ENS may need to be modified when it is applied to irregular structures.

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