

Investigation and Calculation of Seismic Reliability of Structures

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Abstract—Recently, analysis and designing of the structures based on the Reliability theory have been the center of attention. Reason of this attention is the existence of the natural and random structural parameters such as the material specification, external loads, geometric dimensions etc. By means of the Reliability theory, uncertainties resulted from the statistical nature of the structural parameters can be changed into the mathematical equations and the safety and operational considerations can be considered in the designing process. According to this theory, it is possible to study the destruction probability of not only a specific element but also the entire system. Therefore, after being assured of safety of every element, their reciprocal effects on the safety of the entire system can be investigated.

Keywords—Probability, Reliability, Statistics, Uncertainty

I. INTRODUCTION

PREDICTION of seismic response of a new or existing structure is complex, due not only to the large number of Factors that need to be considered and the complexity of Seismic response, but also due to the large inherent uncertainty associated with making these predictions.

Clearly the characteristics of future earthquakes can only be approximated leading to very large uncertainties in the Loads acting on the structure. Structural properties may differ from those intended or assumed by the designer, or

May change substantially during the earthquake (e.g. local fracture of connections). Analysis methods may not accurately capture the actual behavior due to simplifications in the analysis procedure (linear vs. nonlinear for instance) and modeling of the structure. Our Knowledge of the behavior of structures during earthquakes is not complete which introduces other uncertainties. Consequently, seismic performance prediction must consider these uncertainties [8]. Many of these issues are covered to a greater or lesser extent in current codes through the use of Load and resistance factors, of various design Parameters following major earthquakes and introduction of new analytical and design procedures as they are developed and verified.

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By means of the Reliability analysis of the structural systems, it is possible to investigate sufficiency of the structures recently designed and also safety of the existing structures, probability-based analysis of the structure's risks, maintenance strategies, safety evaluation of the wear and tear period etc. Reliability theory enables us to design the structural system through defining the destruction criterion. Hence, one may hope to:

a) Design modern structures without procedural regulations, for example to design a structure made up of new materials

b) Provide the committees, which make decisions about procedural regulations, with this theory as a powerful device and then apply the gained results to substitute and complete the existing procedural regulations

c) Find the contradictions of different procedural regulations and try to remove their deficiencies

Purposes of this paper

1) Structural reliability calculated in two ways : DM-Based and IM-Based , comparing the results

2) Structural reliability calculated using two distribution (Logistic and Log-normal) and comparing results

3) Structural reliability calculated using two (non-linear static and non-linear dynamic) analysis and comparing results.

The main goal of this paper provide fundamental and novel method for calculating probability of the Reliability of structures using statistical distribution is in addition to simply being more efficient, more accurate performance of structures likely to be able to offer us. (Being symmetric is one of the Important characteristics of this distribution which should be noted.)

In this course, different distributions (such as beta, gamma, povason, Laplace, gamble etc.) were examined but not applied due to some specific characteristics like being unsymmetrical.

Logistic distribution was additionally investigated and it was illustrated that this distribution is very much like the Log-normal distribution from the aspect of statistic specifications. The presented mathematics papers are suggestive of this claim, too [16].

II. STRUCTURAL CHARACTERISTIC

The applied model was a reinforced concrete bending frame which has been designed according to the specific plasticity principles and by means of IDARC Non-linear Software. The mentioned frame possesses 8 storeys with 3.2 meters in height and also four 5-meter openings.

To provide the necessary plasticity and meet the economical considerations, the percentage of the columns' armatures and that of the posts were limited to 1 – 3 and 1.7 percents, respectively. The table I contain the dimensions and kinds of the armatures used in the frame's different parts.

In the current research, in order to do the Time history analysis, a series of Record were needed. So, 30 reformed Record were selected. To make sure of being reformed, the entire records were controlled by Seismo Signal software. All the selected records belonged to California, United States and some controlling parameters, such as distance from the fault and largeness, were taken into account while selecting records.

TABLE I
RANGE OF EACH PARAMETER USED TO SELECT RECORDS

Symbol	Quantity	characteristic	SI
S	Soil	B, C, D usgs Classification	-----
M	Magnitude	6---7.3	-----
d	distance	15.1---28.8	1Km → 1000 m
PGA	Peak Ground Motion	0.074 ---0.549	1g → 9.8 m/s ²
P	Number of Point	1800 --- 5961	-----
DT	Duration Time	0.005, 0.01, 0.02 sec	Sec
D	Duration	21.9---40 Sec	-----

III. INCREMENTAL DYNAMIC ANALYSIS

Incremental Dynamic Analysis (IDA) is an emerging analysis method that offers thorough seismic Demand and capacity prediction capability by using a series of nonlinear dynamic analyses under a multiply scaled suite of ground motion records. Realization of its opportunities requires several innovations, such as choosing suitable ground motion Intensity Measures (IMs) and representative Damage Measures (DMs).

An important issue in Performance-Based Earthquake Engineering is the estimation of structural Performance under seismic loads, in particular the estimation of the mean annual rate of exceeding a specified level of structural demand (e.g., maximum Peak interstorey drift ratio θ_{max}) or certain

Needs is Incremental Dynamic Analysis (IDA), which involves Performing nonlinear dynamic analyses of the structural model under a suite of ground Motion records ,

Each scaled to several intensity levels designed to force the structure all the Way from elasticity to final global dynamic instability [14].

Thus, we can generate IDA curves of the structural response, as measured by a Damage Measure (DM, e.g., peak roof drift or maximum peak interstorey drift θ_{max}), versus the ground Motion Intensity level, measured by an Intensity Measure (IM, e.g., peak ground acceleration or the 5%-damped first-mode spectral acceleration $S_a(T_1; 5\%)$).

In turn these can be processed and summarized to get the distribution of demand (DM) given intensity IM. Additionally, limit-states (e.g. Immediate Occupancy or Collapse Prevention [7]) can be Defined on each IDA curve and summarized to produce the Probability of exceeding a specified Limit-state given the IM level.

The Final results are in a suitable format to be conveniently integrated with a conventional hazard Curve In order to calculate Annual rates of exceeding a Certain Limit-state Capacity, or a certain demand.

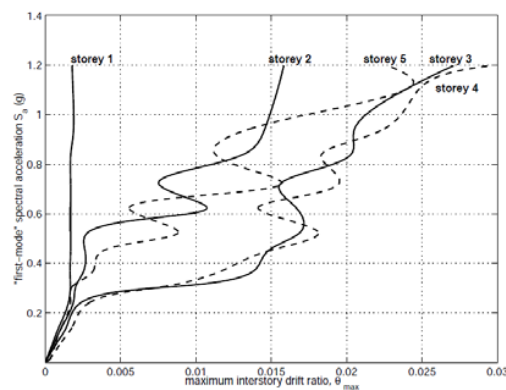


Fig. 1 IDA curves of peak interstorey drifts for each floor of a T1= 1:8, sec 5-storey steel braced frame.

IV. MODAL PUSHOVER ANALYSIS (MPA)

a) Introduction

The basics of this method were proposed by [1], [2]. Applying the concept of one-degree-free structure as well as the desired seismogram allows the equivalent displacement of one-degree-free structure to be achieved, by means of which the displacement of the main structure can be calculated.

b) Implication Equivalent One-Degree-Freedom Structure

The dominant concept in the entire non-linear static analysis methods is the equivalent one-degree-free structure [13]. Indeed, the acceleration displacement response spectrum

(ADRS) in the considered mode is gained through the main structure's capacity curve divided by the modal participation multiplied by the structure's modal amount at roof point ($\Gamma_n \phi_n$) and the normal axis of the main structure's capacity curve divided by the effective modal mass (M_n^*) as well.

Objective of this process in the Capacity Spectrum Method is the simultaneous drawing of demand and capacity parameters in one coordinate system as well as determining their crossing point as the structure's performance point, although in fact obtained curve indicates behavior of the equivalent one-degree-free structure. In this bilinear curve, gradient of the first part reveals behavior of the equivalent one-degree-free structure and gradient of the second part, represented usually as a multiple of the first part's gradient, is indicative of hardening after yield. Having this curve in hand, it is possible to determine un-linear behavior of the one-degree-free structure. Coordinate axes of structure curve are achieved by an un-linear incremental static analysis. First being changed into ADRS, they are transformed into the un-linear behavior of equivalent one-degree-free structure of the unit mass.

It is essential to formulate the movement equation to calculate the structure's response and then solve it. According to the curves shown in Fig. 2, the movement equation of the equivalent one-degree-free structure is presented as "(1)," formula:

$$\ddot{D} + 2\xi\omega_n \dot{D}_n + \frac{F_{sn}(D_n, \dot{D}_n)}{L_n} = -\ddot{u}_g(t) \tag{1}$$

Where, D_n is the displacement response of the equivalent one-degree-free structure, \dot{D}_n and \ddot{D}_n show the first and second derivatives of the selected D_n , respectively, ω_n is the structure's frequency at the n^{th} mode as well as time history of the seismic acceleration. $\frac{F_{sn}}{L_n}$ Can be obtained through the division of normal axis of capacity curve (base shear of the main structure) by the effective modal mass (M_n^*) and \ddot{u}_g is the ground motion.

It is concluded from Fig. 2 that stiffness of the structure is a function of displacement's amount and direction. Through numerically solving the "(1)," or direct modeling of one-degree-free structure with the selected un-linear behavior, maximum displacement of the equivalent one-degree-free structure D_n can be calculated. Thereafter, by means of "(2)," maximum displacement of the main structure's end is obtained:

$$u_{r,n} = \Gamma_n \phi_{r,n} D_n \tag{2}$$

Where, $\phi_{r,n}$ is the deformation of the structure's end at the considered mode and D_n is maximum displacement of the

equivalent one-degree-free structure. Indeed, if authors suppose Capacity Spectrum Method (CSM) as a method based on spectrum analysis, MPA method can be considered as a method based on time history analysis. It is notable that for both mentioned methods, by application of one of the modal combination methods, the results obtained can be generalized to other modes.

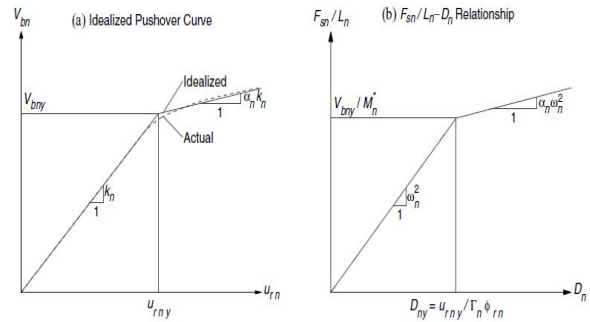


Fig. 2 Properties of the nth-'mode' inelastic SDF system from the pushover curve.

In MPA method, the seismic response of each mode, from the push of building to the target displacement of that mode, is determined by uniform distribution of modal lateral force $S_n^* = m \phi_n$. Since the maximum response of building is obtained through the combination of each mode's seismic response with the appropriate modal combination law, the effects of higher modes would be examined. This method is directly applied to estimate the deformation demand (such as the roof displacement and the relative displacement); however, additional considerations are needed in order to calculate the rotational plastic hinge and elements force. The principal assumptions of this method are un-coupling and super position of the modal responses in the building possessing non-rubber system, i.e. the principal assumptions of non-linear static method.

MPA method allows the seismic demand evaluation to be achieved in two stages [5]:

- a) Execution of several one-modal pushover analyses for different modes so as to determine the matching modal response in the final displacement level.
- b) Then, evaluation of the structure's final response through combining responses of several modes conforming to appropriate modal combination law

V. EARTHQUAKE HAZARD CURVE

Earthquake Hazard curve is indicative of the annual probability of earthquake occurrence according to the intensity level of the probable quake which is different dependent on the site location. In fact, a curve presents number of the annual crossing over special spectrum acceleration for certain T and ξ [4]-[12].

Since achieving the risk curve is beyond the scope of this paper, the data needed to calculate the earthquake risk have been extracted from www.usgs.gov.

In the present investigation, the risk curve of Santa Barbara region, California Province has been applied.

VI. STRUCTURES' SAFETY

Engineers sometimes face failure problems. These failures take place often due to the impact of over-loads, Being Smaller of elements' capacity than the values determined by the design resistance formulas, or the effect caused by their combination. Dictionaries often have defined the term "failure" as shortcoming in accomplishing a task or meeting an expectation. Therefore, definition of the term "failure" will always depend on designing engineer's vision and attitude.

VII. FAILURE PROBABILITY

If stress is considered as the structure's response and symbolized with random variable of S and in the case that we present random variable of R for materials' yield stress (as symbols of demand and capacity, respectively), and the term "failure" is interpreted as stress excess derived from load of materials' yield stress, then failure will occur when [6]:

$$S \geq R \tag{3}$$

In the case that the probability density distribution functions of S and R are normal and independent from each other, so it can be written that:

$$\begin{aligned}
 P(S) &= \frac{1}{\sigma_s \sqrt{2\pi}} \text{EXP} \left[-\frac{1}{2} \left(\frac{s - \bar{s}}{\sigma_s} \right)^2 \right] \\
 P(R) &= \frac{1}{\sigma_R \sqrt{2\pi}} \text{EXP} \left[-\frac{1}{2} \left(\frac{R - \bar{R}}{\sigma_R} \right)^2 \right] \\
 \bar{F} &= \bar{R} - \bar{S} \\
 \sigma_F^2 &= \sigma_R^2 + \sigma_S^2 \\
 P(F) &= \frac{1}{\sigma_F \sqrt{2\pi}} \text{EXP} \left[-\frac{1}{2} \left(\frac{f - \bar{f}}{\sigma_F} \right)^2 \right]
 \end{aligned} \tag{4}$$

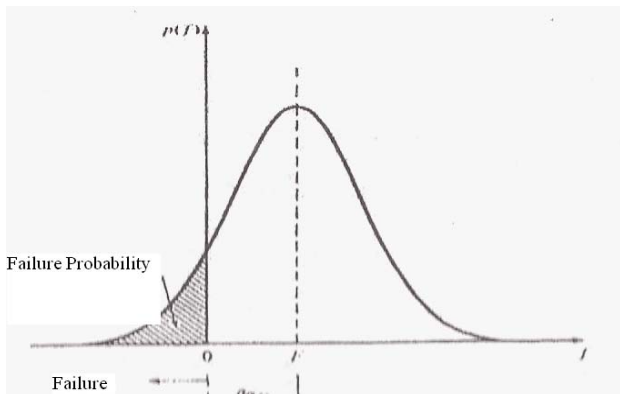


Fig.3 Failure Probability Density Function

Considering Fig.3, the failure probability equals to a probability that $F < 0$, and therefore its value can be obtained from the following "(5),":

$$P_F = P_r(F < 0) = \int_{-\infty}^0 P(F) dF \tag{5}$$

Distance between the coordinate origin i.e. $F=0$ and mean of failure function (\bar{F}) reveals structure's reliability margin and it is usually defined as a coefficient which is exerted on standard deviation of F. it means that:

$$\begin{aligned}
 \bar{F} &= \beta \sigma_F \\
 \beta &= \frac{\bar{F}}{\sigma_F} = \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2 + \sigma_S^2}}
 \end{aligned} \tag{6}$$

Where β is called Coefficient Index Reliability or Safety Index. So, by means of above equations, the statistical parameters of S and R (standard deviation and mean) now explicit and clear, as well as their probability density function, the failure value can be calculated [9]. If density function of load, stress or resistance is logistic, the failure value will be obtained with using the software s-plus and the variable variation "(7),"

$$U = \frac{F - \bar{F}}{\sigma_F} \tag{7}$$

Since the failure probability should be calculated for values of $F < 0$, the value matching $F = 0$ is calculated as described "(8),"

$$U = \frac{F - \bar{F}}{\sigma_F} = \frac{0 - \bar{F}}{\sigma_F} = \frac{-\bar{F}}{\sigma_F} \tag{8}$$

If reliability index is applied according to the above formula, we will have:

$$U = \frac{-\bar{f}}{\sigma_F} = \frac{-\beta \sigma_F}{\sigma_F} = -\beta \tag{9}$$

So, failure probability value is calculated through the following relation:

$$P_r(F \leq 0) = P_r(U \leq -\beta) = \int_{-\infty}^0 P(F) dF = \int_{-\infty}^{-\beta} P(\mu) d\mu \tag{10}$$

VIII. RELIABILITY STRUCTURES THEORY

Reliability Theory, as a branch of Probability Theory, provides us with a firm and logical framework to take account of uncertainty items while calculating capacity and demand.

In general, reliability theory is a scale with the help of which it is possible to assess ability of each part or the whole of an artificial device or system for operating without any failure under any condition considered for it. One of the best definitions of reliability theory presented by NASA is: "reliability means the sufficient (efficient) performance of a system under a predetermined work condition for a certain period of time." The definition shows that reliability is always indicative of a kind of probability which establishes a connection between performance of the system and whatever expected from it in practice.

On the whole, the following stages should first be taken to gain reliability:

- a) Clear demarcation between failure and intactness criteria of the item considered, in other words, offering a precise definition of failure
- b) Selection of a definite model which relates main variables to the failure or intactness criteria.
- c) Detection of uncertainties in main variables
- d) Obtaining probability distribution functions and statistical moments of main variables

When the above mentioned stages are accomplished, it is time to do the analyses needed for achieving reliability.

IX. BASIC PROBABILITY THEORY AND STOCHASTIC VARIABLES

- a) Events and basis probability rules

An event E is defined as a subset of the sample space (all possible outcomes of a random quantity) Ω . The failure event E of e.g. a structural element can be modeled by $E = \{R \leq S\}$ where R is the strength and S is the load. The probability of failure is the probability $P_f = P(E) = P(R \leq S)$. If a system is modeled by a number of failure events, failure of the system can be defined by a union or an intersection of the single failure events [11].

If failure of one element gives failure of the system, then a union (series system) is used to model the system failure, E :

$$E = E_1 \cup \dots \cup E_m = \bigcup_{i=1}^m E_i \tag{11}$$

Where E_i is the event modeling failure of element i and m is the number of events. If failures of all elements are needed to obtain failure of the system, then an intersection (parallel system) is

Used to model the system failure, E :

$$E = E_1 \cap \dots \cap E_m = \bigcap_{i=1}^m E_i \tag{12}$$

Disjoint / mutually exclusive events are defined by

$$E_1 \cap E_2 = \emptyset \tag{13}$$

Where \emptyset is the impossible event.

A complementary event E is denoted defined by

$$E \cap \bar{E} = \emptyset \text{ and } E \cup \bar{E} = \Omega \tag{14}$$

The so-called De Morgan's laws related to complementary events are

$$\begin{aligned} E_1 \cap E_2 &= \overline{\overline{E_1} \cup \overline{E_2}} \\ E_1 \cup E_2 &= \overline{\overline{E_1} \cap \overline{E_2}} \end{aligned} \tag{15}$$

Probabilities of events have to fulfill the following fundamental axioms:

Axiom 1: for any event E :

$$0 \leq P(E) \leq 1 \tag{16}$$

Axiom 2: for the sample space Ω

$$P(\Omega) = 1 \tag{17}$$

Axiom 3: for mutually exclusive events E_1, E_2, \dots, E_m :

$$P\left(\bigcup_{i=1}^m E_i\right) = \sum_{i=1}^m P(E_i) \tag{18}$$

The conditional probability of an event E_1 given another event E_2 is defined by:

$$P(E_1 / E_2) = \frac{P(E_1 \cap E_2)}{P(E_2)} \tag{19}$$

Event E_1 is statistically independent of event 2 E if

$$P(E_1 / E_2) = P(E_1) \tag{20}$$

From "(19)," we have

$$P(E_1 \cap E_2) = P(E_1 / E_2) P(E_2) = P(E_1) P(E_2) \tag{21}$$

Therefore if E_1 and E_2 are statistically independent:

$$P(E_1 \cap E_2) = P(E_1) P(E_2) \tag{22}$$

Using the multiplication rule in "(21)," and considering mutually exclusive events E_1, E_2, \dots, E_m the total probability theorem follows:

$$\begin{aligned} P(A) &= P(A / E_1) P(E_1) + P(A / E_2) P(E_2) + \dots + P(A / E_m) P(E_m) \\ &= P(A \cap E_1) + P(A \cap E_2) + \dots + P(A \cap E_m) \end{aligned} \tag{23}$$

Where A is an event.

From the multiplication rule in "(21)," it follows

$$P(A \cap E_i) = P(A / E_i) P(E_i) = P(E_i / A) P(A) \tag{24}$$

Using also the total probability theorem in "(23)," the so-called Bayes theorem follows from:

$$P(E_i / A) = \frac{P(A / E_i) P(E_i)}{P(A)} = \frac{P(A / E_i) P(E_i)}{\sum_{j=1}^m P(A / E_j) P(E_j)} \tag{25}$$

- b) Continuous stochastic variables

Consider a continuous stochastic variable X . The distribution function of X is denoted $F(x)$ and gives the probability $F_X(X) = P(X \leq x)$. A distribution function is illustrated in Fig.4. The density function $f(x)$ of X is illustrated in Fig.4 and is defined by

$$f_X(X) = \frac{d}{dx} F_X(X) \tag{26}$$

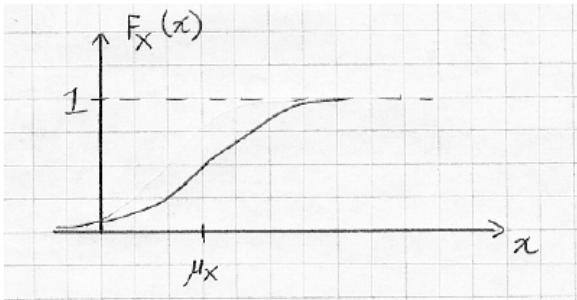


Fig. 4 Distribution function $F(x)$ of X .

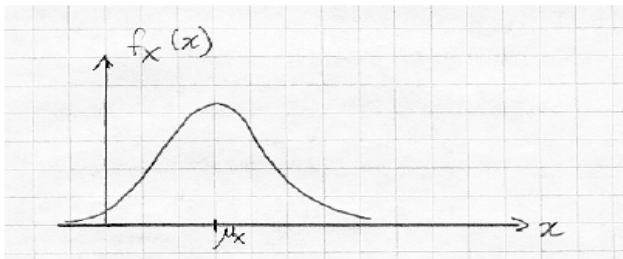


Fig. 5 Density function $f(x)$ of X .

The expected value is defined by

$$\mu = \int_{-\infty}^{+\infty} x f_X(x) dx \tag{27}$$

The variance σ^2 is defined by

$$\sigma^2 = \int (x - \mu)^2 f_X(x) dx \tag{28}$$

Where σ is the standard deviation.

The coefficient of variation $COV = V$ is

$$V = \frac{\sigma}{\mu} \tag{29}$$

The n^{th} order central moment is

$$m_n = \int (x - \mu)^n f_X(x) dx \tag{30}$$

The skewness is defined by

$$\beta_1 = \frac{m_3}{m_2^3} \tag{31}$$

And the kurtosis is

$$\beta_2 = \frac{m_4}{m_2^2} \tag{32}$$

X.PROBABILITY OF FAILURE FUNDAMENTAL CASE

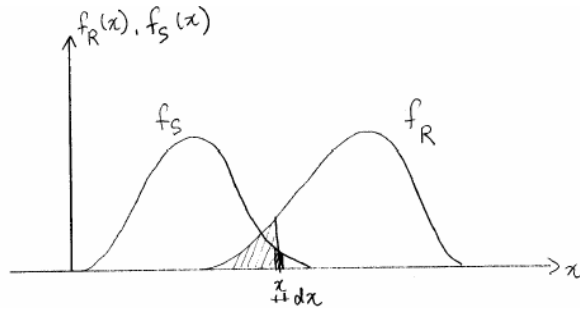


Fig. 6 Density functions for fundamental case.

Consider a structural element with load bearing capacity R which is loaded by the load S . R and S are modeled by independent stochastic variables with density functions f_R and f_S and distribution functions F_R and F_S , see Fig. 6 The Probability of failure becomes

$$P_f = P(\text{failure}) = P(R \leq S) = \int_{-\infty}^{+\infty} P(R \leq x) P(x \leq S \leq x + dx) dx \tag{33}$$

$$= \int_{-\infty}^{+\infty} F_R(x) f_S(x) dx$$

Alternatively the probability of failure can be evaluated by

$$P_f = P(\text{failure}) = P(R \leq S) = \int_{-\infty}^{+\infty} P(x \leq R \leq x + dx) P(S \geq x) dx \tag{34}$$

$$= \int_{-\infty}^{+\infty} f_R(x) (1 - F_S(x)) dx = 1 - \int_{-\infty}^{+\infty} f_R(x) F_S(x) dx$$

For Example: Logistic distribution

The distribution function for a stochastic variable with expected value μ and standard deviation S is denoted $N(\mu, s)$, and is defined by

$$F_X(x) = L\left(\frac{x - \mu}{s}\right) = \int_{-\infty}^x \frac{e^{-\left(\frac{t - \mu}{s}\right)}}{s \left(1 + e^{-\left(\frac{t - \mu}{s}\right)}\right)^2} dt \tag{35}$$

Where $L(u)$ is the standardized distribution function for a Normal distributed stochastic variable with expected value = 0 and standard deviation = 1 : $N(0,1)$

The Logistic distribution has:

Skewness: $\beta_1 = 0$

Kurtosis: $\beta_2 = \frac{6}{5}$

XI. SEISMIC STRUCTURAL RELIABILITY CALCULATION

a) DM – BASED

The new procedure can be used for new design, evaluation of existing buildings and evaluation of damaged buildings after an earthquake. It is a performance based procedure with two limit states considered, Collapse Prevention and Immediate Occupancy.

This paper will deal only with Collapse Prevention. The design object is to have 90% confidence that the chance of not Satisfying limit state is less than 2% in 50 (2/50) years.

The seismic hazard level for the performance limit is also chosen to be 2/50. The acceptance criterion is based on a confidence factor, λ , that is used to determine the confidence level.

This factor is the ratio of the factored demand over factored capacity. In equation form, this is expressed as:

$$\lambda = \frac{\gamma \cdot \gamma_a \cdot D}{\phi \cdot C} \quad (36)$$

Where

D = estimate of median drift demand

C = estimate of median drift

Capacity

ϕ = Resistance factor

γ = Demand factor

γ_a = Analysis demand factor

The factors, ϕ , γ and γ_a in “(36),” are based on the reliability work developed by [10] for the SAC project.

A more detailed derivation of these equations is given by [3].

“Equation (36) is essentially the ratio of factored demand divided by factored capacity.” The demand, D , is the expected median drift resulting from a series of accelerograms sampled from the chosen hazard level.

Details on how to calculate all of the variables for this procedure is given in [15].

The resistance factor, ϕ , accounts for the fact that the estimate of C is affected by randomness and uncertainty in the estimation process. The capacity of the building against global collapse is a function of the earthquake accelerograms used in the IDA analyses [14]. These accelerograms are part of a random process. The capacity is also affected by the uncertainty in the load-deformation behavior of the system determined from tests. The local

collapse value is also affected by uncertainties in the response of the components due to variable material properties and fabrication.

The “(37),” for calculating the resistance factor,

ϕ is given by [3] :

$$\phi = \phi_{RC} \cdot \phi_{UC} \quad (37)$$

$$\phi_{RC} = e^{\frac{-k\beta_{RC}^2}{2b}} \quad (38)$$

$$\phi_{UC} = e^{\frac{-k\beta_{UC}^2}{2b}} \quad (39)$$

Where

ϕ_{RC} = Contribution to ϕ from randomness of the earthquake accelerograms

ϕ_{UC} = Contribution to ϕ from uncertainties in measured component capacity

The demand factor, γ , is calculated as:

$$\gamma = e^{\frac{k}{2b}\beta_{RD}^2}$$

$$\gamma_a = C_B \cdot e^{\frac{k}{2b}\beta_{UD}^2} \quad (40)$$

Where

C_B : Precision parameter analysis of nonlinear dynamic analysis of mode value is 1.0 and the nonlinear static analysis mode, its value is 0.997.

$\beta_{RD} = \sqrt{\sum \beta_i^2}$ Where β_i^2 is the

Variance of the natural log of the drifts for each element of uncertainty.

The confidence factor, λ depends on the slope of the hazard curve, k and the uncertainty, but not randomness, associated with the natural log of the drifts. The “(38),” for λ is [10]

$$\lambda = e^{-\beta_{UT}(k_x - k\beta_{UT}/(2b))} \quad (41)$$

Where

$\beta_{UT}^2 = \sum \sigma_i^2$ Where σ_i is for

Uncertainties in the demand and capacity but not randomness

k = slope of the hazard curve

K_x = standard Gaussian variate associated with probability

x of not being exceeded

$$u = \frac{\ln(\lambda) + \frac{k}{2b} \cdot \beta_{UT}^2}{\beta_{UT}} \quad (42)$$

U = Logistic Parameter

Calculate the probability of reliability of seismic structures using probability distribution Logistic

$$RE = 1 - L(u) = 1 - \frac{e^{-u^2}}{(1 - e^{-u^2})^2} \quad (43)$$

RE = Reliability

$L(u)$ = Logistic distribution

b) IM – BASED

This Method

$$\lambda = \frac{C_B \cdot D_{IM}}{\phi \cdot C_{IM}} \quad (44)$$

In Which

$$\phi = e^{-0.5 \cdot k \cdot \beta_{IM}^2 \cdot c} \quad (45)$$

Also

$$u = \frac{\ln(\lambda) + \frac{k}{2} \cdot \beta_{UT}^2}{\beta_{UT}} \quad (46)$$

Other steps of the process of calculating the reliability of similar structural seismic is DM – Based

Final notices:

Overall reliability of structural systems can increase capacity in extreme cases or reducing uncertainty in capacity, improved. The average capacity can be increased using stable configuration, the system increases, stronger and harder to components such as fittings Reload against excessive fatigue failures caused by having low cycle, can be provided. This

increase causes less response at every level and thus reduce the likelihood of extreme state is.

XII. CONCLUSION

According to curves presented in Appendix section can be concluded

a) The reliability structure curves Fig.7 and Fig.8 can be concluded that the structural reliability calculation method based on the DM using the log normal probability distribution in the low intensity higher S_a shows high intensity, but this revealed Logistic probability distribution is likely to show more confidence. This point can be concluded that values derived from logistic probability distribution values more reliable because the high intensity, structural stability will be further considered.

b) “Fig.” 9 and Fig.10 show the method is based on DM structure reliable than methods based on the IM shows can be reliability method-based IM calculation made, and an estimate conservative reliability of structural seismic acquired .

c) According to Fig. 11 and Fig.12 is clear that the structural reliability calculation method based on analysis of DM using IDA more likely than the MPA analysis will result.

Notice:

According to the authors of many studies on the distribution Logistic did the conclusion that the Logistic distribution expressed a higher probability of occurrence of the actual distribution is log normal [16].

APPENDIX

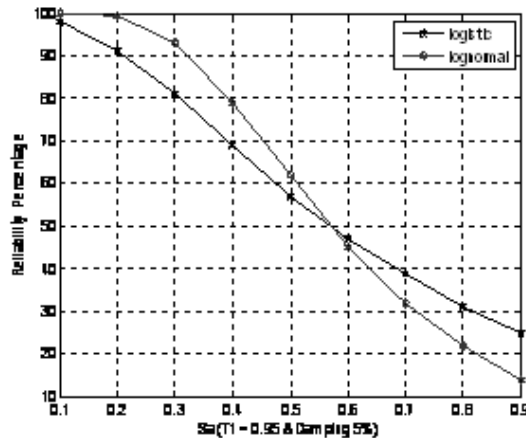


Fig.7 Reliability Plot Compare between Logistic and Log-normal Distribution

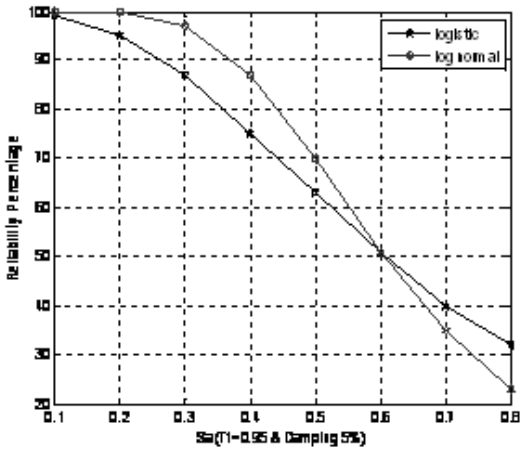


Fig.8 Reliability Plot Compare between Logistic and Log-normal Distribution

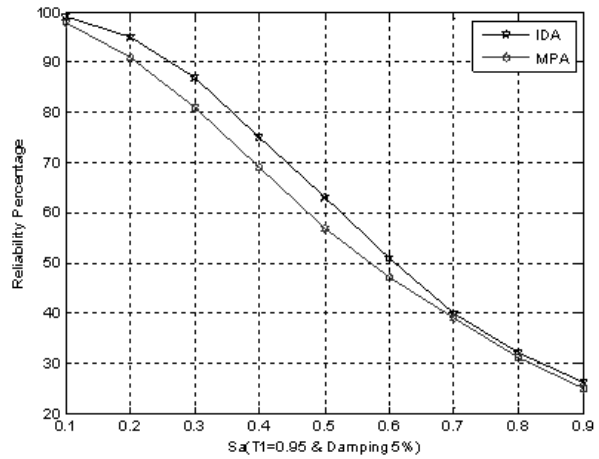


Fig.11 Reliability Plot Compare between IDA and MPA analysis by Logistic distribution

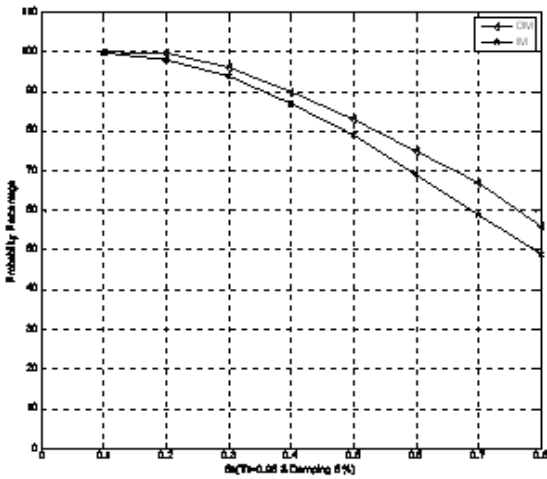


Fig.9 Reliability Plot Compare between DM-Based and IM-Based

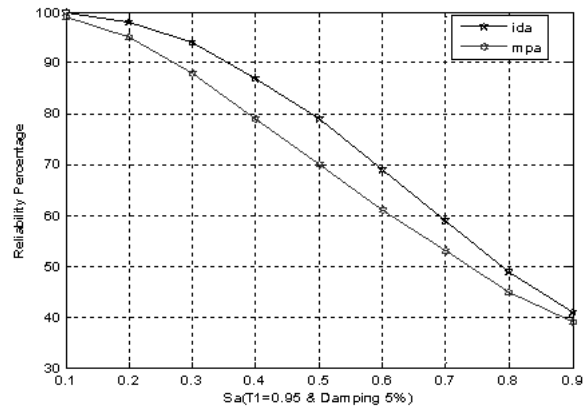


Fig.12 Reliability Plot Compare between IDA and MPA analysis by Logistic distribution

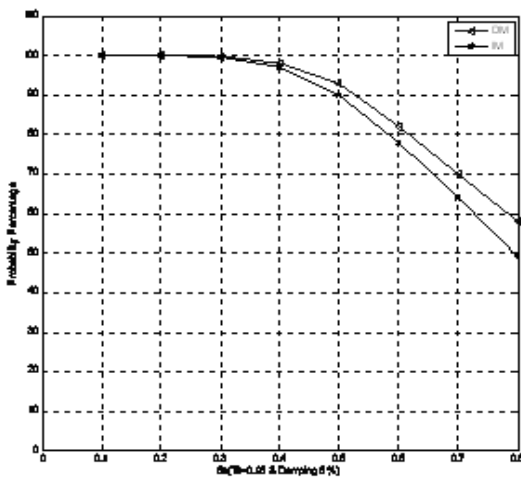


Fig.10 Reliability Plot Compare between DM-Based and IM-Based

REFERENCES

- [1] Chopra,A.K.,Goel , R.K.(2001).” A modal pushover analysis procedure for estimating seismic demand for building.theory and preliminary evaluation “. [Report No.PEER-2001/03].Pacific Earthquake Engineering Research Center,University of California , Berkeley
- [2] Chopra,A.K.,Goel , R.K.(2002).” A modal pushover analysis procedure for estimating seismic demand for building.theory and preliminary evaluation “.J. Earthquake Engineering and Structural Dynamics,31,561-582.
- [3] Cornell, C. Allin, Jalayer, Fatemeh, Hamburger, Ronald O., and Foutch, Douglas A. (2002). “The probabilistic basis for the 2000 SAC/FEMA steel moment frame guidelines”, ASCE Journal of Structural Engineering, Vol. 128, No.4, pp 526-533.
- [4] DOE (1994). “Natural phenomena hazards design and evaluation criteria for Department of Energy Facilities”, DOE-STD-1020-94, U. S. Dept. of Energy, Washington, D. C.
- [5] Donald E.Grierson ,yanglin Gong , Lei Xu (2006) “ Optimal performance based seismic design using modal pushover analysis “ journal of Earthquake Engineering , Vol.10,No.1(2006) 73-96
- [6] Doris LLOYD Grosh(1989)”A Primer of Reliability Theory” john Wiely &Sons ,Inc
- [7] FEMA 350 (2000). “Recommended seismic design criteria for new steel moment-frame buildings.” Report No. FEMA-350, SAC Joint Venture, Federal Emergency Management Agency, Washington, DC.

- [8] Foutch,D.A(1999),”Performance Prediction and Evaluation of Steel Moment Frames Under Seismic Loads.”SAC joint Venture Report No. XX , FEMA XXX.
- [9] Hari,G.C.(1982)”Uncertainty Analysis ,Loads , and Safety in Structural Engineering , Prentice-Hall , Inc. Englewood Cliffs , N.J.
- [10] Jalayer.F. and Cornell,C.A.(1998),” Development of a Probability-Based Demand and Capacity Factor Design Seismic Format”Draft.8.11.98.Publication as a SAC Background Document Pending.
- [11] John Dalgaard Sørensen(2004)” Structural Reliability Theory And Risk Analysis” Institute of Building Technology and Structural Engineering Aalborg University Sohngaardsholmsvej 57, DK-9000 Aalborg, Denmark
- [12] Luco, N.; Cornell, C. A. (1998) “Seismic drift demands for two SMRF structures with brittle connections”, Structural Engineering World Wide 1998, Elsevier Science Ltd., Oxford, England, 1998, Paper T158-3.
- [13] Valley, M.T., Harris, J.R.(1998).” Application of model techniques in a pushover analysis”.6th US National Conference on Earthquake Engineering.
- [14] Vamvatsikos, D. and Cornell, C. A. (2001). “Incremental dynamic analysis”, Earthquake Engineering and Structural Dynamics. 31(3), pp 491-514.
- [15] Yun, S.Y., Hamburger, R. O., Cornell, C. A., and Foutch, D. A. (2002). “Seismic performance evaluation for steel moment frames”. ASCE Journal of Structural Engineering, Vol. 128, No.4, pp 534-545.
- [16] Raminta Stockute, Andrea Veaux, Paul Johnson (2006). “Logistic distribution”