Assessment of the Seismic Behaviour of the Reinforced Concrete Structures Based on the Probabilities: Implementing Fragility Curves

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Abstract- **The response of reinforced concrete structures to earthquake is a complicated issue that has been the subject of researchers' studies for years. At present, various methods are used to evaluate the seismic resistance of existing buildings in the world. These methods are established in such ways that they only have an application in different types of reinforced concrete buildings constructed in a country. Therefore, direct application of them in other countries is not possible. On the other hand, design control of the existing reinforced concrete structures and determining their behaviour against earthquake motions, needs a rather easy applied method. One of the main desired goals on this basis is to introduce a new method for computing the probability of vulnerability of reinforced concrete structures that besides simplicity, and accuracy in computation, presents a more realistic and pragmatic probability of structures vulnerability and also can readily compute the level of vulnerability of other structures and be able to present in valid regulations of seismic design of structures, because it seems that it expresses the Log normal distribution probability very conservatively. Therefore, Logistic probability distribution was studied and it appeared that it has the ability of computing probability of vulnerability of the structures with high accuracy. In this paper, the attempts have been made to apply the logistic distribution simultaneously with the Log-normal distribution and to compare their results.**

Keywords- Vulnerability; Seismic Design; Reinforced Concrete; Logistic Distribution; Probability

I. INTRODUCTION

In recent years many studies and researches have performed in order to estimate the risk level of structures. Available records and documents show startling statistics of evaluation of economic damages in seismic areas and therefore in order to reduce the potential of damage that depends on seismic performance of structures, the amount of damages incurred to structures should be predicted by valid and reliable methods. Among introduced methods is IDA (Incremental Dynamic Analysis) Curves that are obtained through nonlinear dynamic analysis of the structure under earthquake accelerograph. Structure's response such as displacement, relative drift of stories, accelerations and so on is determined by numerous nonlinear dynamic analyses in which maximum acceleration of earthquake increase continuously. This trend will continue until instability of structure and then the resultant response will be drawn against PGA of earthquake. These curves show useful information about risk level of structures. Two other methods can be used for describing structural damage amount in a given area [5]:

1. Fragility Curves,Damage.
2. Probability Matrix1.

Damage Probability Matrix is indeed an indication of damages distribution in form of table. Each column is considered for one earthquake intensity and numbers in columns show a fraction of different ranges of damages experienced by structures. The sum of numbers in one column is 1. DPM shows the discrete probability of incidence of one damage range for different intensities. On the other hand, fragility curves provide graphical information of structural damage distribution. Therefore, information provided by DPM and fragility curves are similar and the only difference is that fragility curves show a cumulative distribution of damages that suggests the continuous probability of incidence or crossover of a damage range. These curves are of specific importance. Generalities and concepts of fragility curves are described in this paper.

II. SEISMIC FRAGILITY

Description of seismic risk involves three major parameters:

- Earthquake risk,
- Characteristics of structure components,
- Fragility of structure components with regard to seismic risk.

During the process of providing information about fragility, inherent uncertainties in risk and structure component should be introduced in calculations and integrated in uncertainties due to calculation methods. Fragility curves give us an estimation of structure crossover from a limit state in a particular level of intensity. Limit state usually gives a damage state or applied limitation in terms of structure response. In recent studies, limit states are expressed as terms of transformation instead of load. Fragility information can be used by design engineers, reliable specialists, insurance specialists and managers of crisis systems, hospitals and highways network. This information can be used for analysis, evaluation and improvement of seismic performance of structural and non-structural systems. Also, various sources of data can be used for achieving fragility curves.

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III. DAMAGE INDEX

Damage index is a response quantity or a dimensionless ratio of structure response to earthquake simulation that is used as measure for criterion for measuring imposed damages

1 -Whithman 1973

of structures based on performance ranges. Damage indices are defined in local and total forms (as shown in Fig. 1).

Fig. 1 Damage index

Two types of the oldest and the most simple damage indices are plasticity and relative drift of stories. Plasticity ratio can be in form of curvature (μ_ϕ), rotation (μ_θ) or displacement (μ_δ).

$$\mu_\phi = \frac{\phi_m}{\phi_y}, \mu_\theta = \frac{\theta_m}{\theta_y}, \mu_\delta = \frac{\delta_m}{\delta_y} \tag{1}$$

Where δ_m, θ_m, ϕ_m are maximum curvature, maximum rotation and maximum displacement at the end of one component, respectively, and δ_y, θ_y, ϕ_y are yield curvatures, yield rotation and yield displacement, respectively. Banon (1981) improved plasticity ratio based on hardness reduction and resistance reduction that occurs in oscillatory loading. Bending damage ratio is defined in the following way [4]:

$$FDR = \frac{K_0}{K_m} \tag{2}$$

Where K_0 is initial scant hardness and K_m is the scant hardness equivalent to maximum response and k_f is final scant of final hardness. Comparison with experiments results shows that it does not have any of plasticity ratios and failure or damage compatible results. Another quantity that has been used widely as a damage index is relative drift of stories. The fact that stories drift is used for measuring expected damages is generally accepted. Algan (1982) and Hazus (2003) used relative drift as a basis for measuring damages due to earthquake.

IV. LOCALIZED INDICES BASED ON DEFORMATION

Stephen and Yao (1987), Wang and Shah (1987), Wang (1992) and Chung (1987) suggested a different damage index for modeling bulk damages which occur in oscillatory loading. This damage index was implemented either by relation making based on fatigue in which damages were considered as a function of localized plastic deformations or by combining a part of energy absorbed during loading [10].

V. COMPOUND INDICES

Willums and Sexmith (1995), Gouzman (1989) and Valles (1996) described Park & Ang Index which is the most famous and the most widely used localized damage index. This index is a linear combination of normalized deformations and absorbed energy [6, 9].

$$D = \frac{\delta_m}{\delta_u} + \beta_e \frac{\int dE}{F_y \delta_u} \tag{3}$$

Where D is damage index, δ_m is maximum change response under earthquake, dE is absorbed hysteretic energy component, δ_u is final deformation capacity under uniform deformation, F_y is yield resistance of longitudinal reinforcements, and β_e is a positive constant coefficient that represents effect of oscillatory loading on structural damage. This coefficient is a function of shear span ratio, normalized axial stress, longitudinal steel ratio and closure ratio of steel. Park (1985) suggested D = 0.4 as threshold value of repairable and non-repairable damages, while some researchers in 1987 presented a more detailed classification which is shown in Fig. 2.

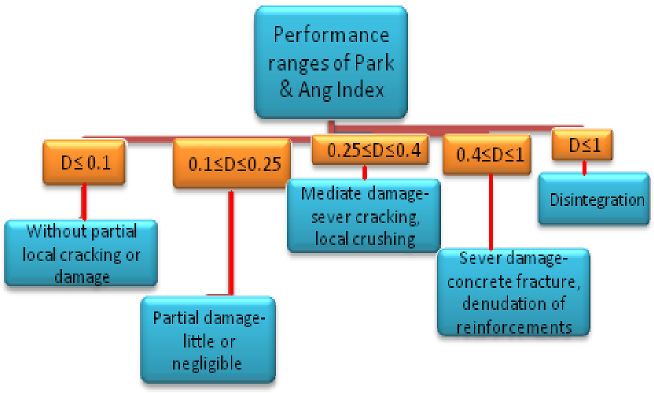

Fig. 2 Performance ranges of Park & Ang Index

VI. TOTAL DAMAGE INDEX

Total damage index of a structure depends on distribution and intensity of local damages. Method widely used for expressing total damage index of a structure is the weighted or normal average of local indices or local absorbed energy. Therefore, for a story:

$$D_{story} = \frac{\sum D_i E_i}{\sum E_i} \tag{4}$$

Where D_i is local damage index in i^{th} point and E_i is energy absorbed in i^{th} point. Natural frequencies of damaged structures reduce due to reduction of hardness. Roufaiel and Meyer presented a total damage index with a correction factor [8]:

$$D_{global} = \frac{\delta_m - \delta_y}{\delta_f - \delta_y} = \frac{14.2\delta_y\left(\sqrt{\frac{f_{und}}{f_{dam}}} - 1\right)}{\delta_f - \delta_y} \tag{5}$$

Where F_{und} and F_{dam} are structure foundation frequencies before and after damage, δ_f is final capacity and δ_y is yield capacity under uniform loading. A number of softening indices, such as maximum softening D_{ms}, are calibrated based on three periods shown in Fig. 3.

$$D_{ms} = 1 - \frac{T_{und}}{T_m}; D_{ps} = 1 - \frac{T_{dam}^2}{T_m^2}; D_{fs} = 1 - \frac{T_{und}^2}{T_{dam}^2} \tag{6}$$

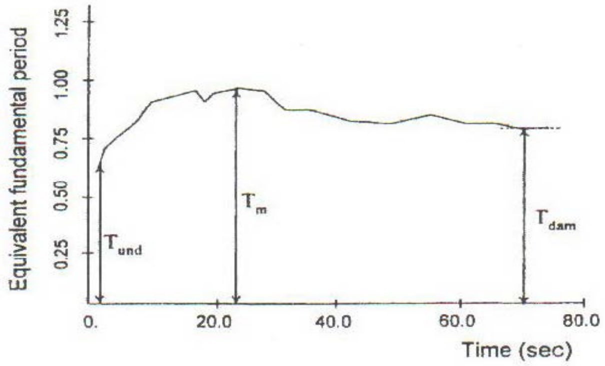

Fig. 3 Basic period evolution of Milican Laboratory (Willimas and Sexsmith, 1995)

Where T_{und} and T_{dam} are structure periods before and after earthquake and T_m is the maximum period during earthquake. Regarding to these indices, maximum softening was suggested as the best quantity for showing total damage index of structure. Merk (1992) extended the maximum softening index in order to include second mode. He defined two damage parameters [7]:

$$D_1 = 1 - \sqrt{\frac{K_{1,m}}{K_{1,und}}} \tag{7}$$

$$D_2 = 1 - \sqrt{\frac{K_{2,m}}{K_{2,und}}}$$

Where K_1, K_2 are spring constants for a system of two freedom degrees with two equal masses. Its periods are equivalent to first and second periods of multi-freedom degree structure. It can be assumed that D_1, K_1 represent lower part of the structure and D_2, K_2 represent the upper part of the structure.

VII. DRAWING FRAGILITY CURVES BY SIMPLE METHOD

As mentioned before, structure fragility for a limit state is defined as conditional probability of transition from limit state capacity for a given level of ground motion intensity. So it can be said that knowing demand values for different levels of spectral acceleration (Incremental Dynamic Analysis data) and knowing limit state capacity, we can determine probability of transition from given limit state at a certain level of intensity by following formula :

$$P_{LS} = \frac{n_{data/(data>LS)}}{n_{Total-data}} \tag{8}$$

In this method, we do not take a certain statistical distribution for data and all the inherent random characteristics will be effective directly. But a disadvantage of this method is that it does not have a formula definition for damage function.

VIII. PROVISION OF FRAGILITY CURVES BY INTENSITY-BASED METHOD

Structural fragilities for a limit state are defined as conditional probability of transition from limit state capacity for a given intensity level [11] (as shown in Fig. 4)

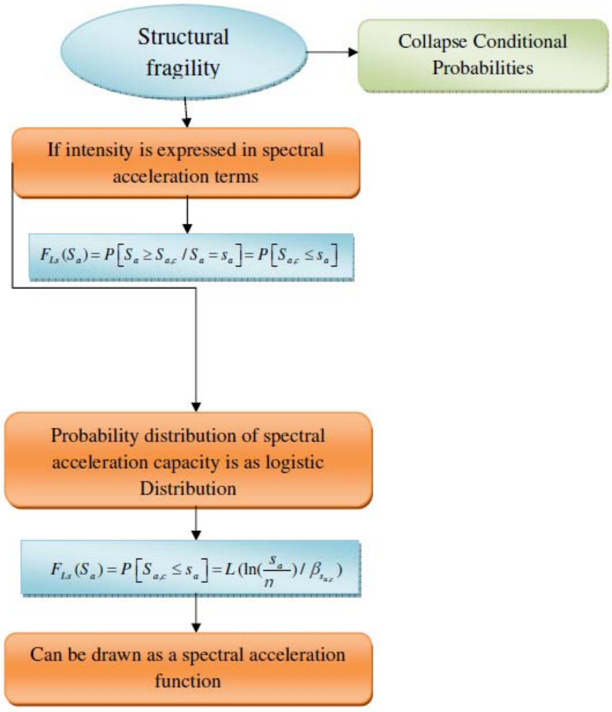

Fig. 4 Structural fragility flowchart

Where $F_{Ls}(S_a)$ is structure fragility at spectral acceleration S_a for limit state Ls. From above expression it can be found that fragility is expressed as probability that random variable $S_{a,c}$ be lower than or the same as S_a value. So, fragility is still expressed as cumulative distribution functions of random capacity $S_{a,c}$ [11]. If it is assumed that probability distribution of spectral acceleration capacity, $S_{a,c}$ is as logistic log with mean $\eta_{S_{a,c}}$ and natural log standard deviation $\beta_{S_{a,c}}$. Schematic fragility curve is showed in Fig. 5 [11].

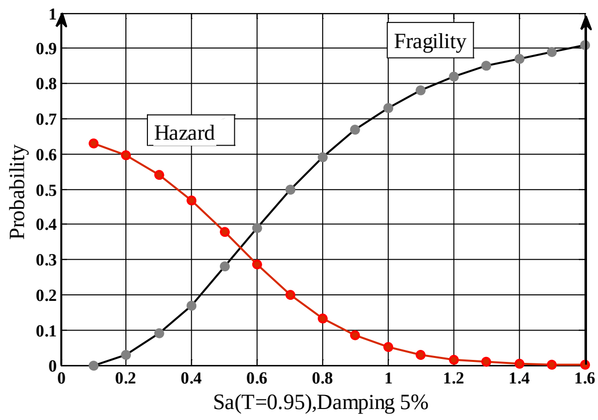

Fig. 5 Schematic diagram of fragility and risk curve

IX. FRAGILITY CURVE USING 3D DAMAGE INDEX

In constructing analytical vulnerability functions, damage state of a structure is estimated based on analytical assessment results such as maximum displacements, interstory drifts and various damage indexes. From the latter analysis results, probability distribution and its parameters are determined. However, if the demand is far larger than the capacity of the structure, the analysis often stops due to the failure of numerical convergence in the algorithm of the structural analysis program. This incomplete analysis may result in very large response value, as shown in Fig. 6. The latter analysis result should be excluded in the statistical estimation of structural responses because it distorts the statistical parameters such as mean and standard deviation that represents the damage state of structures [12].

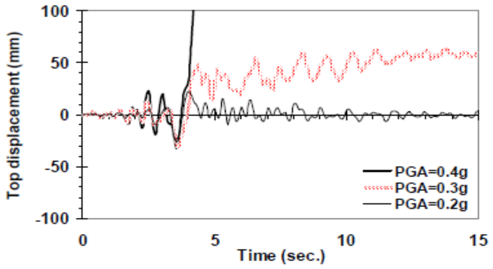

Fig. 6 Response histories of the example structure under various intensities of Kalamate 1986- Prefecture

In this paper, analysis results with D.I. larger than 1.0 are separated from the procedure of determining statistical parameters to estimate the level of structural damage and the probability of collapse during the analysis was calculated, as shown in Step 1(Fig. 7). For structures that are not collapsed during the analysis ($0 \leq$ D.I. ≤ 1.0), conventional method of calculating cumulative probability distribution is multiplied with the probability of non-collapse which is the complementary probability of collapse[12], as shown in Step 3 (Fig. 7). Finally, the probability of damage index exceeding a predetermined limit state is calculated by combining the probability of collapse and the probability of structures suffering a certain level of damage without collapsing. Similar concept to the above mentioned method is given by Shome and Cornell (2000). For a given earthquake intensity, the procedure of calculating the probability of damage index exceeding a given limit state (L.S.) is described as follows [12]:

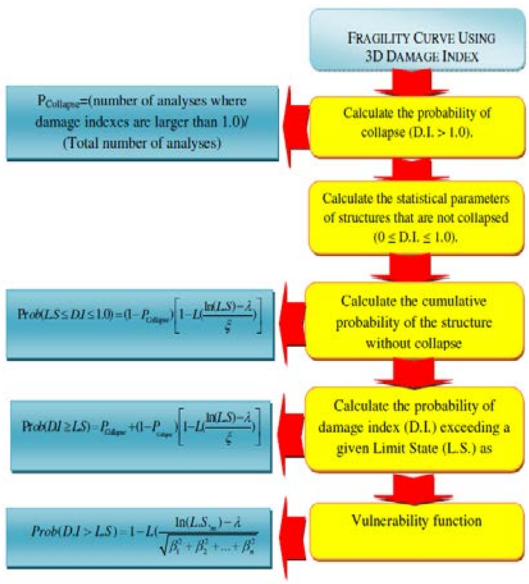

Fig. 7 Fragility curve using 3D damage index flowchart

The parameters used are defined as follows:

μ: Mean of Damage Indexes (D.I.) that are between 0 and 1.0

σ: Standard deviation of Damage Indexes (D.I.) that are between 0 and 1.0

$\delta = \frac{\sigma}{\mu}$: Coefficient of variation, $x_m = \frac{\mu}{\sqrt{1+\delta^2}}$: Median

$\lambda = \ln x_m$, $\xi = \sqrt{\ln(1+\delta^2)}$ and $L(...)$ cumulative standard Logistic distribution.

The probability distribution is assumed to be Logistic for convenience. Fig. 8 shows the comparing of fragility curves by lognormal distribution and logistics distribution.

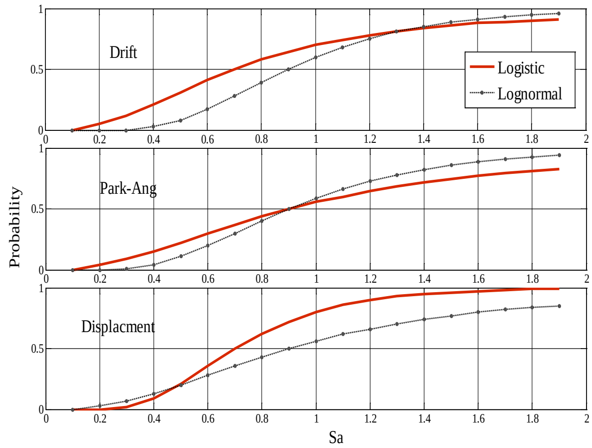

Fig. 8 Comparison of fragility curves by logistic distribution and lognormal distribution

Vulnerability functions using the new 3D damage index and conventional damage index are compared in Fig. 9. Four Limit States are defined based on damage indices D.I. of 0.2, 0.4, 0.6 and 0.8. Since the structure is under bi-directional loading, conventional damage index is defined as the bi-directional combination of unidirectional demand-to-capacity ratio which is the ratio of maximum displacement demand to the ultimate displacement capacity, as follows [12]:

$$D.I.(SRSS) = \sqrt{\left(\frac{\Delta_{\max,x}}{\Delta_{Ultimate,x}}\right)^2 + \left(\frac{\Delta_{\max,y}}{\Delta_{Ultimate,y}}\right)^2} \quad (9)$$

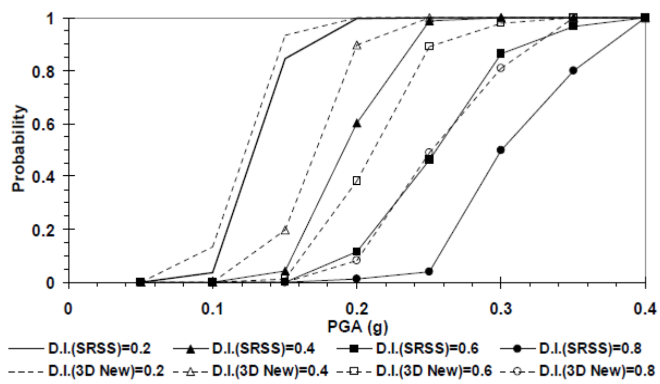

Fig. 9 Comparison of vulnerability functions using 3D damage index and conventional Damage Index

Fig. 9 clearly shows that vulnerability functions using the 3D damage index represents higher risk than those using the conventional damage index. The difference between two types of vulnerability functions becomes larger as the limit state represents more significant damage level [12].

Where L.S.m is the median value of a displacement limit state and βi (i=1, 2, …, n) represent various uncertainties [12]. These can be demand uncertainty, capacity uncertainty and modeling uncertainty as in (Wen et al., 2004) or simply response and capacity uncertainty (Dimova and Hirata, 2000). Modeling of uncertainty is not presented in this paper, though its importance in seismic loss assessment cannot be over-emphasized.

X. PROVISION OF FRAGILITY CURVE BY POWER-LAW METHOD

To determine fragility curve, Power-Law can be used [2, 3]. To this end, first non-collapse probability should be obtained from following equation:

$$P(NC) = a.(Sa)^{b} \tag{10}$$

a and b are spectral acceleration curve parameters obtained from following expression:

$$a = \text{Empirical}F(LS) / Sa^{b} \tag{11}$$

$$b = \ln(\frac{Sa_1}{Sa_2}) / \ln(\frac{\text{Empirical}F(LS)_1}{\text{Empirical}F(LS)_2}) \tag{12}$$

S_a is spectral acceleration and NC is non-collapse state.

It should be noted that to determine a and b parameters, Empirical F (Ls) need to be determined which is obtained from following equation:

$$\text{Empirical}F(LS) = (1-[\sum_{i=1}^{30} n_i \ / 30]) \tag{13}$$

n is the number of data which had reached to collapse state in a given acceleration. In this method a limit state is defined that whenever structure crosses this state collapse will happen. Sa_0 is taken as limit spectral acceleration and β is considered as partial standard deviation. Both are obtained from following expressions:

$$\beta = -b$$
$$Sa_0 = Exp(\ln(a)/\beta) \tag{14}$$

If $S_a \geq S_{a0}$ means that the structure has crossed the given limit state, collapse has happened, therefore fragility probability is obtained from following equation [1]:

$$S_a \geq S_{a_0} \Rightarrow F(LS) = 1 - P(NC) \tag{15}$$

LS is the limit state.

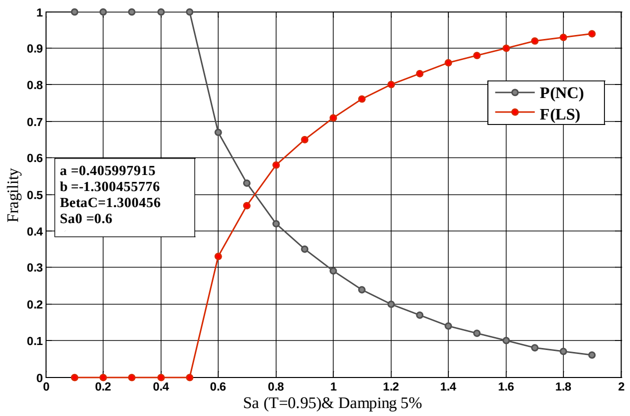

Fig. 10 Compare fragility curve (Power Law Method)

XI. CONCLUSION

According to investigations conducted by the author determined that the Log normal probability distribution, the probability of the vulnerability of structures to be very conservative calculation. According to the research was clear that the likely vulnerability of structures using the Log normal distribution approximately 20% of the structures more vulnerable than the Logistic distribution shows (Figs. 11 and 12), the fact considers less risk, but designing structures in discussion of economic problems and desirable structural safety with regard to reasonable risk of the most essential things. Therefore it is suggested to calculate the probability of structural vulnerability of the Logistic distribution probability distribution simple, symmetrical, and the likelihood and ability to accurately calculate the probability of occurrence and the response it has caused (Bader taking some reasonable risks) instead of Log normal probability distribution use.

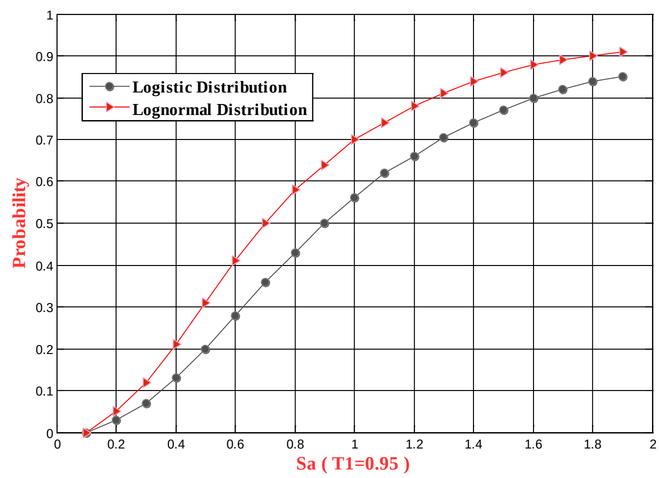

Fig. 11 Compare fragility curve logistic & lognormal distribution – drift IDA

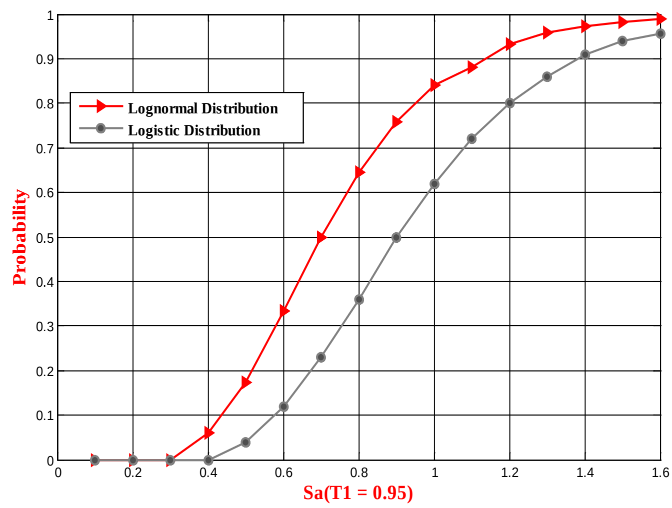

Fig. 12 Compare fragility curve logistic & lognormal distribution- displacement IDA

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