Studying the Seismic Reliability in the Cable-Stayed Bridge

Mehdi Vajdian[1], Alireza Habibi[2] & Ali Parvari[3]

[1] Faculty member, Department of CIVIL ENGINEERING, Aligodarz Branch, Islamic Azad University, Aligodarz, Iran

[2] Faculty member, Department of CIVIL ENGINEERING, Arak Branch, Islamic Azad University, Arak, Iran

[3] Faculty member, Department of CIVIL ENGINEERING, Khomein Branch, Islamic Azad University, Khomein, Iran

Correspondence: Mehdi Vajdian, Faculty member, Department of CIVIL ENGINEERING, Aligodarz Branch, Islamic Azad University, Aligodarz, Iran. E-mail: esfehani.mohamad3@gmail.com

Abstract

With regard to the abundant importance of bridges and the need to study the behavior of these structures against earthquake forces and also new and strong methods for analysis of structures in this article, the probabilistic structure based on seismic reliability is used in order to assess the seismic performance of the members of a cable bridge by using of incremental non-linear dynamic analysis. This analysis method is an efficient tool for estimating the need and capacity in engineering probabilistic method according to the performance. With regard to the unpredictable and uncertain property of earthquake and the hypotheses existing in modeling the bridges and uncertainties existing in demand and capacity and with regard to the losses and severity parameters, the method based on reliability is used for studying in these structures according to the intended performance levels. In this research, the purpose is to study the seismic reliability of one of the cable bridges which has been designed according to the seismic criterion of Iran country (Ahvaz). In this research, sap2000 software has been used for incremental non-linear dynamic analysis, and the reliability level is calculated with regard to FEMA351 instruction.

Keywords: cable stayed bridge, incremental non-linear dynamic analysis

1. Introduction

From the view of structure engineering, the cable-stayed bridges have notable features such as non-linear behavior in large opening moths of this kind of structures. In the road and bridge regulations in countries like Japan and America, these kinds of structures are analyzed and designed in terms of earthquake, wind, traffic load and rainfall. But non-linear behavior of these structures in large earthquakes highlights the need to more accurate study and seismic loading effect on them. Anyway, correct use of cable bridges in seismic areas requires enough awareness of the earthquake effect on this kind of structures. Effectiveness or ineffectiveness of the earthquake loading and its effect degree on designing the structure require accurate and comprehensive analyses of these structures influenced by seismic loads and correct recognition and estimation of it. One of the cases which hasn't been studied about cable bridges is the seismic reliability of the cable-stayed bridges. Day-increasing progress of data process technology in computer sciences has provided the possibility of incremental analyses, the same issue has cased that the analyses start from static and elastic state and progress to the elastic dynamic analyses and after it non-linear static analyses and finally non-linear dynamic analysis. In non-linear dynamic analysis, it was necessary that several different records to be enforced for each time study that mostly due to the voluminous work, this method is used for controlling the accomplished designs. With similarity in passing from the linear static analysis method to non-linear static analysis method, this idea was created that we should be propelled from linear dynamic analysis method to the incremental dynamic analysis method, in a form that seismic load to be scaled and increased gradually. The concepts of this method have been expanded in 1977 by Bertero and in recent years, the designing method according to the capacity and demand which had been developed by Allin Cornell, have been considered by many researchers and with regard to have the ability to be expressed in the probabilistic frame, it can be used for determining the reliability level and improvement of the structures performance. In 1991, Abdel-Gaffar & et al presented their researches in the field of seismic structure of three

dimensional cable bridges, in this research the seismic behavior of cable bridges in three dimensional state was studied and non-uniform movements of the earth and consequently non-uniform and uniform forces exerted on the piers were considered (Nazmy & Abdel-Ghaffar, 1990; Andani, Moghaddam, Haberland, Dean, Miller & Elahinia, 2014; Hadi, Qasemi, Elahinia & Moghaddam, 2014). Designing in the seismic field according to the displacement of this kind of structures and seismic performance was done in 2001 by Park et al, in 2004 by Mo & Nien, in 2005 by Luo et al and in 2007 by Nielsen & Des Rouches (Endo, Kawatoh & Unjoh, 2004). In these researches, designing ascends according to the design and construction cost performance while the cost of repair after expected earthquake descends.

2. Data and Material

Generally, non-linear behavior is originated from three main sources, the past researches have studied these three factors (Nazmy & Abdel-Ghaffar, 1990):

1-Non-linear behavior arising from diagonal cable deflection due to the existence of distributed load arising from their weight.

2-Non-linear behavior arising from interaction between axial loads and bending anchor in the bending members of structure.

3-Non-linear behavior arising from changes in the structure geometry due to the large deformations which can occur under the service designing loads.

In this research, three above factors have been considered in modeling. For non-linear behavior of diagonal cables which are influenced by the deformation effect of these members, we use of an equivalent elasticity modulus for the direct member. The rigidity of diagonal cables to high extent depends on the axial force existing in the cable and its special weight. In a manner that if the amount of force existing in the cables is small, the cable rigidity will be little and with increasing of the axial force of cable, its rigidity is also increased. This method for the first time was presented in 1965 by a person named Ernest. The work is in this form that the equivalent elasticity modulus is obtained from the following formula and it is replaced instead of elasticity modulus of flat cable:

$$Eq = E/(1+(AE(WL)^2/12T^3)$$

Two other factors can be exerted in sap2000 software easily. The materials behavior is also assumed non-linear.

The studied model in this research is in Ahvaz(figure 1) on the largest river which is being constructed. The specifications of this bridge are as follows:

1-It has two concrete towers with height of 81 meter.

2-It has 12 concrete piers on both sides of the bridge.

3-The main opening moth of bridge is 212 meter.

4-Deck at the outset of opening mouth is from concrete and in central opening mouth, it is from open metal section.

5-Each cable has formed from 27 strands and each strand has formed from 7 steel wires with nominal diameter of 15.7mm with final tensile stress of 1860 Mega pascal. Cables follow the Europe standard. In the figures 2, 3, 4, the sections are observed well.
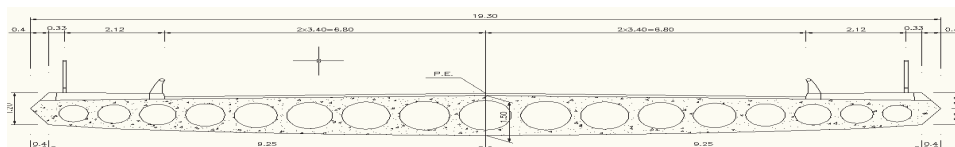

Figure 1. Cable-stayed bridge
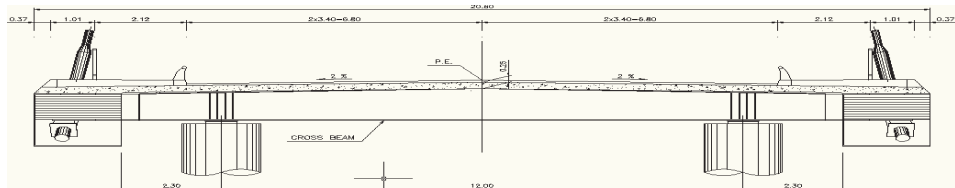
Figure 2. Concrete section of the bridge deck

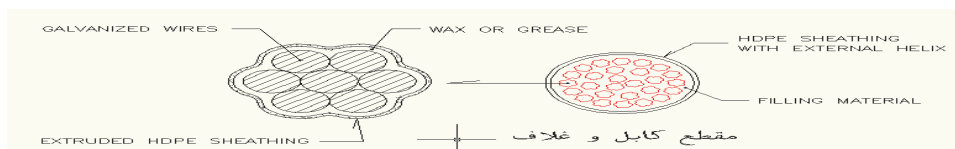Figure 3. Metal section of the bridge deck

Figure 4. Cable section

3. Research Methodology

For proper modeling of deck that its section is from two different genera, the best solution is to convert the deck to the concentrated masses and then the seismic behavior of bridge should be studied, but this equivalent-making should be in a manner that the behavior of each component of deck to be close to the reality at the time of earthquake waves hitting. Wilson and Gravel in 1991presented Pie model (figure 5) that according to it, two concentrated masses in both sides of the deck element create distortion of deck which causes combined bilateral and torsion movement (Ghobarah, 2001; Moghaddam, Elahinia, Miller & Dean, 2014; Rahmanian, Moghaddam, Haberland, Dean, Miller & Elahinia, 2014). The above model seems proper for the deck with open metal section and U-boot concrete sections. Modeling the deck has been accomplished by using of the suggested model of Wilson and Gravel that we model the deck with two concentrated masses in sap2000 software. As it was explained, in this model which is applied more for metal and open decks, the main girders which are usually two items and are placed in X-cables, in longitudinal direction are divided into several masses that each mass is exactly under the linkage place of deck and cable, the distance of the concentrated mass with the deck is half of the height of deck section, therefore two masses are placed on both sides of deck, thus we allocate the dead load of deck to the above concentrated masses. Then two concentrated masses are connected to two massless rigid elements. Also in longitudinal direction, a beam element in the deck level connects throughout the deck to each other. This element has a section equal to the deck section but without dead load. In addition, the inertia moment of longitudinal section is also given to the beam element (spine). In this research, deck has been divided into the parts with approximate length of 12 meter, then the dead weight of deck in the length of 12m is calculated and it is divided between two related masses with equal portion. In the table 1-3, you observe a sample of above loading for different sections of bridge. Cables modeling has been done by using of truss elements which act only in tension, and the linkage of cable to the deck and piers has been assumed in joint linkage form. For modeling the piers and tower, bar element with designed sections is used. The specified concrete strength is 35 Mega pascal which has been assumed in clamped form and in the wall under the deck due to the elastic behavior of soil, a series of linear spring is used (Chopra, 2001; Shayesteh, 2015; Esfahani, Andani, Moghaddam, Mirzaeifar & Elahinia, 2016; Moghaddam, Skoracki, Miller, Elahinia & Dean, 2016).
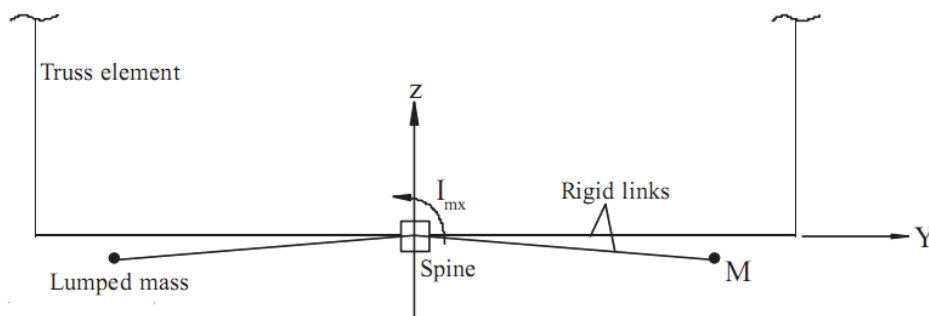
Figure 5. Theory of modeling the deck-suggested model of Wilson-Gravel

Table 3-1. Situation and loading the concentrated masses of bridge deck

Deck section	Concentrated mass of M_1	Concentrated mass of M_2	Vertical distance of masses from deck	Horizontal distance of masses from deck center
Metal	490.3325 k.newton	490.3325 k.newton	1 meter	9.3 meter
Concrete	2353.59 k.newton	2353.59 k.newton	0.6 meter	9.3 meter

With regard to the instruction of FEMA351 for incremental analyses in order to calculate the reliability levels, 10 to 20 accelerograms are studied. In this research, 10 accelerograms have been used in the domain near the fault. On the other hand, these accelerograms should have long alternation time, since cable-stayed bridges with large opening moths have large alternation periods. For this purpose, accelerograms that their A/V ratio is equal or less than one, are selected. Ten intended accelerograms which have been adapted from PEER site, have been specified in the table 3-2. And in the figure (6), elastic design spectrum with 3% damping has been drawn. Then for incremental dynamic analysis, we increase from 0.001g.PGA to 1g. In the continuation, the failure index is placed on horizontal axis and severity index is placed on vertical axis and IDA diagram is drawn. In this research, the severity index is PGA and failure index of maximum relative shift is on top of one of towers.

Table 3-2. Earthquake record

| NO	Earthquake record	PGA (g)	PGV (cm/s)	PGD (cm)	Closest to fault rupture	A/V
1	NORTHR/NWH360	0.59	97.2	38.05	7.1	0.6
2	NORTHR/NWH090	0.583	75.5	17.57	7.1	0.76
3	SUPERST/B-ICC000	0.358	46.4	17.5	13.9	0.77
4	SUPERST/B-ICC090	0.258	40.9	20.2	13.9	0.63
5	SUPERST/B-WSM090	0.172	23.5	13.0	13.3	0.73
6	SUPERST/B-WSM180	0.211	31.0	20.3	13.3	0.68
7	TABAS/TAB-LN	0.836	97.8	36.92	-------	0.85
8	TABAS/TAB-TR	0.852	121.4	94.58	-------	0.704
9	IMPVALL/I-ELC270	0.215	30.2	23.91	8.3	0.71
10	KOBE/TAZ090	0.694	85.3	16.75	1.2	0.81
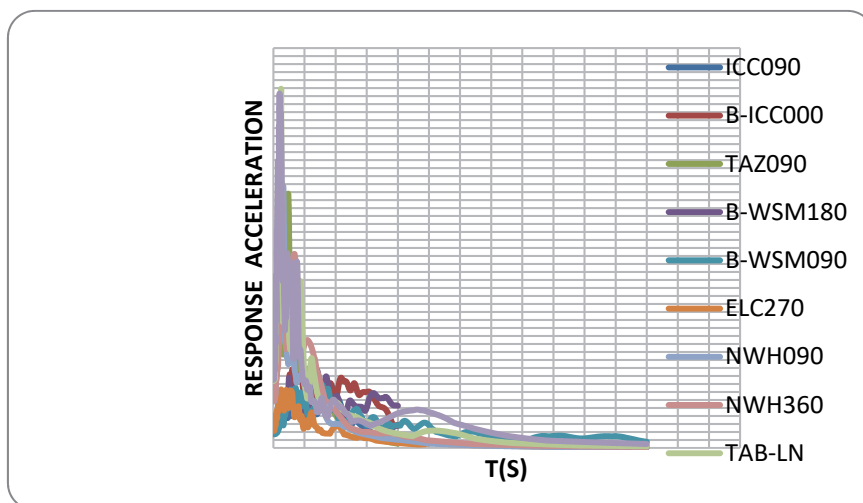
Figure 6. Elastic design spectrum

4. Reliable Theory in FEMA351

In earthquake engineering, accurate modeling of structural systems behavior influenced by probable earthquakes stimulations in future for the designing purposes is difficult and complex. This difficulty is due to the uncertain and unpredictable movements of earth in a region, hypotheses used in analyzing and designing the structures, shortage of knowledge in modeling the structural behavior especially in the non-elastic response area in repeated load cycles and uncertain models. Generally uncertainties in this method are divided into two groups (Chen & Choi, 2001):

1-Uncertainties that reflect the changeability of the result in the repeated experiences and they are recognized explicitly by a random model.

2-Uncertainties which have been the result of the shortage of design knowledge and they exist in the model and its parameters. Some of these parameters are: non-structural components effect, non-classic damping, real hysteretic behavior of systems and real dimensions of the structure.

Under the FEMA/SAC program, the total probability theory was used for assessment of the performance of steel bending frame system. Application of this theory is for recognizing the conditions of designing that keeping the failure probability in mathematic frame is defined in the following form (Kim & Chen, 2000; Raad, Moghaddam & Elahinia, 2016):

$$P_{PL}=P(C≤D)=\int P(C \leq D|D)=d_i)|dH_D(d)| \tag{4-1}$$

that the failure probability or in other words the probability of increasing of failure from the performance level has been considered and $= d_i \int P(C \leq D|D)$ is the conditional probability of increasing of its structure demand or in other words, it is the conditional probability of the structure failure with considering the specified severity (d_i) of earthquake that the structure will experience. In order to simplify the reliability analysis method in the structures for engineers᾽ use, this method has been expressed in the FEMA-350 & FEMA-351 instructions in parametric form. The simplified relation of reliability parameter of FEMA (λ) is expressed in the following form:

$$\lambda = \frac{\gamma\gamma_a D}{\phi C} \tag{4-2}$$

γ: it indicates increase in demand which depends on the hypotheses existing in modeling and prediction of the earth movement specifications, and it is obtained from the following relation (the factor of increase in demand):

$$\gamma = e^{\frac{k\beta^2_{(D|s_a)}}{2b}} \tag{4-3}$$

$_a\gamma$: it is uncertainty factor in the analysis that due to use of special methods for estimation, the structure need is

created as a function of the earth movement severity.

.ϕ: it is the reducer factor of capacity which is obtained from the following relation:

$$\Phi = e^{\frac{k\beta_C^2}{2b}} \qquad (4\text{-}4)$$

D & C are in order the representatives of capacity and demand which are the middling of the statistical population obtained from Incremental Dynamic Analysis (IDA).

With calculation of the needed uncertainty parameters from the relations (3-4) and (4-5), determining the demand and capacity of the structure is obtained from 4-1 relation by using of non-linear dynamic analyses of time history and incremental non-linear dynamic analysis of reliability parameter. After determining the reliability parameter of λ and by using of the following relation, the reliability level can be calculated.

$$k_x = \frac{k\beta_{UT}}{2b} - \frac{\ln\lambda}{\beta_{UT}} \qquad (19\text{-}5)$$

in which k_x is the Gaussian standard variable and β_{UT} is the dependent parameter on the changeability and uncertainties of the total structure in estimation of its demand and capacity and it is calculated from the following relation:

$$\beta_{UT} = \sqrt{\sum \beta_i^2} \qquad (20\text{-}5)$$

By using of K_x parameter and the assumption of lognormal distribution of demand and capacity, the amounts of the structure reliability level in the performance purpose can be determined from the normal distribution tables directly.

5. Results and Analyses

For determining the reliability levels, determining the performance level in the failure index is needed. With regard to this issue that DRIFT in this research has been defined as the failure index, therefore with regard to the reference (6) in the table 5-3, the performance level has been specified (Ren & Obata, 1999).

Table 5-3. Performance level and maximum displacement

Relative displacement of the upper point of tower	Damage states
DRIFT<=0.2%	No damage
DRIFT<=0.5%	Slight damage
DRIFT<=1.5%	Repairable damage
DRIFT<=2.5%	Extensive damage
DRIFT>2.5%	Complete damage

In this article, the reliability in the second performance level has been calculated. In the continuation in the table 5-4, we observe twenty cases in a modal analysis, then with analysis of non-linear time history, you can observe IDA diagram in the figure 7 and finally according to FEMA351 instruction, the reliability parameters are determined.
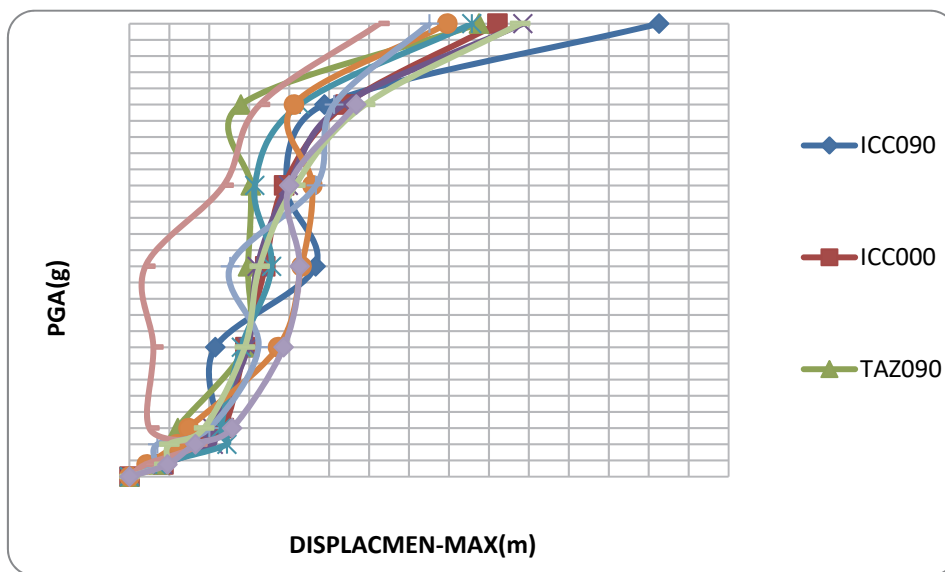
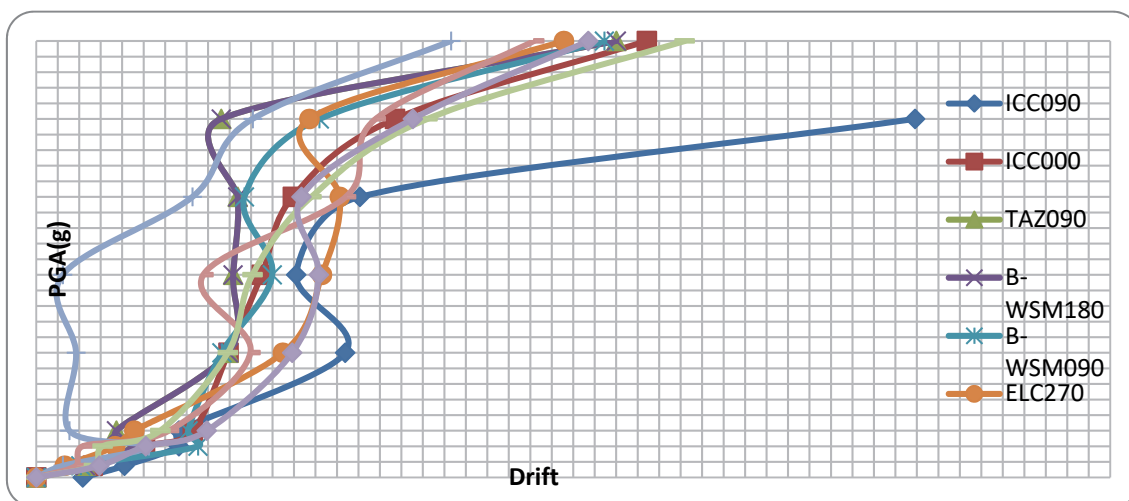Figure 7. IDA diagram according to the maximum displacement

Figure 8. IDA diagram according to the maximum Drift

Table 5-4. Calculation of reliability parameters

Sa(T*)	HD(d)	Ppl	Phee	Gama	Gama*D	Phee*C	Landa	Check	Kx
0.03	2025.943	0.047557	0.85918	1.140568	0.080347	0.472183	0.17016	Adequate	5.042
0.08	190.7727	0.062426	0.858225	1.230595	0.189812	0.471658	0.402436	Adequate	2.775
0.12	69.2807	0.066777	0.857272	1.250125	0.227384	0.471134	0.482631	Adequate	2.369
0.32	4.117644	0.059164	0.856319	1.20288	0.345761	0.470611	0.734707	Adequate	1.468
0.52	1.345316	0.07451	0.855367	1.27998	0.396509	0.470088	0.84348	Adequate	1.197
0.72	0.295077	0.040833	0.854417	1.075585	0.403775	0.469565	0.85989	Adequate	1.057
0.92	0.162916	0.045395	0.853467	1.105027	0.514636	0.469043	1.097202	Adequate	0.435
1.12	0.073017	0.036007	0.852519	1.032902	0.874662	0.468522	1.866854	NOT	-1.322

6. Conclusions

1. The method of incremental non-linear dynamic analysis for full perception of the response domain of structure for different severity levels of earthquake is a very efficient and proper method. Also the changes in deformation models and the outset of the procedure related to the reduction of rigidity and resistance are seen in the diagrams well and dynamic capacity can be estimated relatively more accurate.

2. The response of a structure to a series of accelerograms can be fully different and this issue specifies the importance of statistics and probabilities in engineering.

3. With regard to the diagrams of curve distribution and maximum deformations of the bridge towers, it can be concluded that the talent of formation of primary plastic joint in the height range of 5 to 15m of the earth level is more than the other points that this range is close to the deck.

4. IDA curves of Cable Bridge indicate this issue that in general state in this kind of structures in more severe earthquakes, the performance level of collapsing is seen less than other kinds of structure.

5. Comparing the results of IDA curves under accelerograms indicates that the outset of non-linear phase of above Cable Bridge occurs in PGAs less than 0.1g.

6. Comparing the results of reliability analysis for the cable bridge in the performance level of collapsing indicates that if we consider the acceptance criterion according to FEMA, the structure in PGAs more than 0.92 g will not provide the intended performance level. The reliability level is less than 90 percent.

7. In three dimensional comparison, it can be said that if we define the performance level in the second level, the structure in PGAs more than 0.92g doesn't provide the above performance level again that approves the result No.6.

8. IDA curves of structures like building, according to what has been indicated in the past researchers have rigidity and softness states. In this structure, the need of above behavior is observed but much more.

9. The collapsing threshold for the towers A of the figure with fan system with regard to IDA curves and obtained reliability levels can be searched in PGA of 1.12g.

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