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FRAGILITY CURVES OF AN EXISTING RC BUILDING

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ABSTRACT

Fragility functions of existing buildings play a fundamental role in seismic risk mitigation policies. The latest methodology developed by the authors and their co-workers for estimating direct losses from earthquakes in reinforced concrete (R/C) buildings are presented; they concern the derivation of capacity curves and vulnerability (fragility) curves in terms of peak ground acceleration (PGA), as well as spectral displacement, for all types of R/C buildings that are common in Albania.
The vulnerability assessment methodology is based on the hybrid approach, which combines statistical data with appropriately processed results from nonlinear dynamic or static analyses that permit interpolation and (under certain conditions) extrapolation of statistical data to PGAs and/or spectral displacements for which no data is available.
A detailed discussion of the limitations of the hybrid approach is provided, along with a proposal for improving the quality of results by applying a weighting technique to both the analytical and the statistical input data.
In this paper, a procedure to develop analytical fragility curves for Moment Resisting Frame Reinforced Concrete building is presented. The design of the selected building typologies was performed according to the codes at the time of construction using force-based methods and the state of the practice at the time of construction.

INTRODUCTION

Recent European earthquakes (e.g., Southern Italy 1980, Turkey 1999, L'Aquila 2009) have shown that structural performance of Reinforced Concrete (RC) buildings (in particular Moment Resistant Frame, MRF) always play crucial roles in terms of earthquake losses. Mainly due to the high vulnerability of the building stock, the resilience of the communities has been generally non-existent. Framed RC structures are commonly found in many countries. They represent approximately 75% of the building stock in Turkey, approximately 60% in Colombia, and over 30% in Greece. Moreover, in several past studies, the significant presence of RC MRF buildings in the Balkans building stock has been noted. Consequently, in Albania and other Mediterranean earthquake-prone countries, the seismic performance of the building stock needs to be investigated. Much works have already been done regarding understanding seismic risk and its mitigation , to allow simple and optimised rules for practical planning (support to decision maker) and design to be defined. Simple, fast, available, and economic retrofitting strategies should be defined and integrated in mitigation policies for non-seismic buildings. In this way, it is also possible to increase the resilience of cities in a short time after earthquakes. Tools specifically defined for emergency management and seismic risk mitigation policies must be defined. Examples of these tools are the Vulnerability Index and Fragility Curves (FCs) for building typologies, based on numerical analyses, to study the vulnerability and possible retrofitting. Generally, these methods should be applied using a significant amount of data related to their characterisation, which can be obtained by historic failures, expert evaluation, and field survey or investigation. It is the opinion of the author that the derivation of FCs from post-earthquake or expertise data cannot be sufficient for the realisation of a reliable risk assessment tool. In fact, even a validation for a similar area with surveyed damage may not be sufficient to extend the obtained results. Thus, on the basis of this comparison, the provided results might be grossly misleading. On the other hand, the comparison with past events

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may not be useful to the forecast for the future. On the contrary, this approach must be supplemented by numerical analyses. Generally, a newly proposed method should be capable, on the one hand, to ensure sufficient reliability, and the preparation of seismic scenarios should not be too costly; on the other hand, the newly proposed method should ensure a realistic prevision of the structural performance of the studied buildings to define accurate, large-scale retrofitting policies. Due to the importance of the topic, a considerable number of studies were funded, developed and published in the last years in order to define FCs. These studies are based on different analysis methods and procedures. They are generally refereed to several typologies of RC structures built in a single country.

In this paper, a simple building type with RC Frame is selected and analysed. Moreover, the structural response of the building is expected to be characterised for significant uncertainties. As widely reported in previous studies the variability (randomness) of the seismic action plays a fundamental role in the variability of the structural response. This is particularly the case for Mediterranean RC buildings and their seismic performance assessments with regard particularly to hazard analysis, response damage , and loss analysis. Generally, a good procedure to define the expected damage is a mechanics-based approach. On the basis of deformation demands, the result of the procedure should be physical damage to the structure. Damage levels could also be defined based on repair costs. In any case, generally these methods should be applicable only when using a significant amount of data related to their characterisation, which can be obtained by historic failures, expert evaluation and field survey or investigation. The derivation of FCs from post-earthquake or expertise data (e.g., empirical data) cannot be sufficient for the realisation of a reliable risk assessment. On the contrary, this approach must be supplemented by numerical analyses. Although these approximate limits can seem fairly reasonable, it is clear that for each building type and each territorial application, the specific limits should be defined. Thus, the fundamental step of the present work, in order to achieve the proposed objective, is the correct definition of the relationship between damage level and damage status defined through accurate non-linear analyses. This objective has a fundamental role in the assessment of the seismic capacity of existing construction, in post-earthquake emergency management and in experimental activity (numerical or in the laboratory). Each damage level should be quantitatively established through limit values of local demand parameters on structural members. Then, the correlation between local and global failure can be established. The present work would be an additional contribution along the complex path to seismic risk reduction through the improvement of the seismic vulnerability assessment of RC MRF buildings.

MATERIALS AND METHODS

Case study

Technical characteristics of the RC-building

Considered structure is a part of a building complex in Taftalidze-557, Skopje, Macedonia.
The whole complex, including the considered structure, has foundation, ground floor and eleven stories. Height of a ground floor is 3.06 m, the height for ten upper stories is 3.06 m, the height of the roof floor is 2.3 m. Total height of the building is 35.96 m.

Plan of the building is almost rectangular (dimensions 21.7/15.9 m), consisting of frames in both the X and Y directions.

Total floor area is 306.9 m^2, area of roof floor is 122.43m^2, while total area of the building is 11*306.9 m^2+122.43 m^2 = 3498.33 m^2 (see figure 1).

The structural system for resisting loads in longitudinal X direction consists of 6 frames of type R1X,R1X1, R1XP,R1X1P,R2X,R2XP (more details of geometry are given in Figure 2).

The structural system for resisting loads in lateral Y direction consists of 6 frames of type R1Y, R1YP, R2Y, R2YP, R3Y, R3YP (more details of geometry are given in Figure 2).

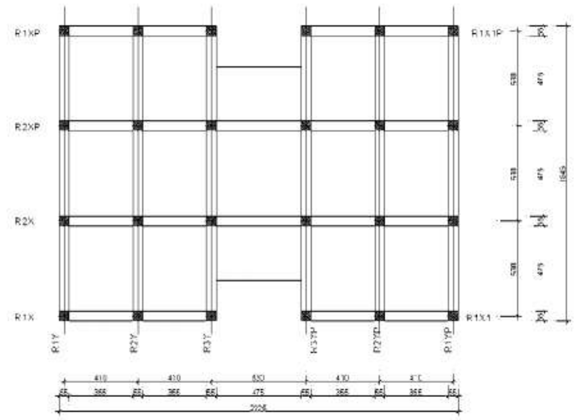
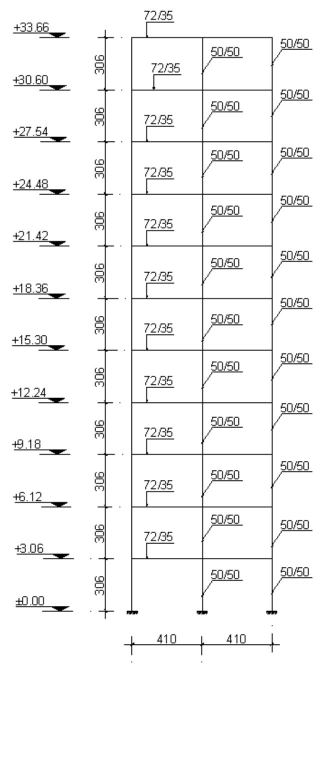
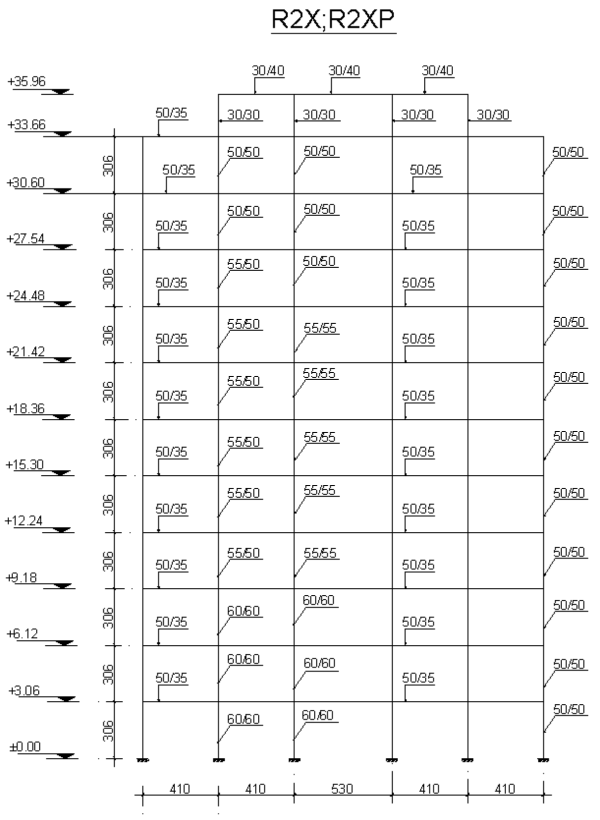


Figure 1 Typical floor

R1X;R1X1;R1XP;R1X1P





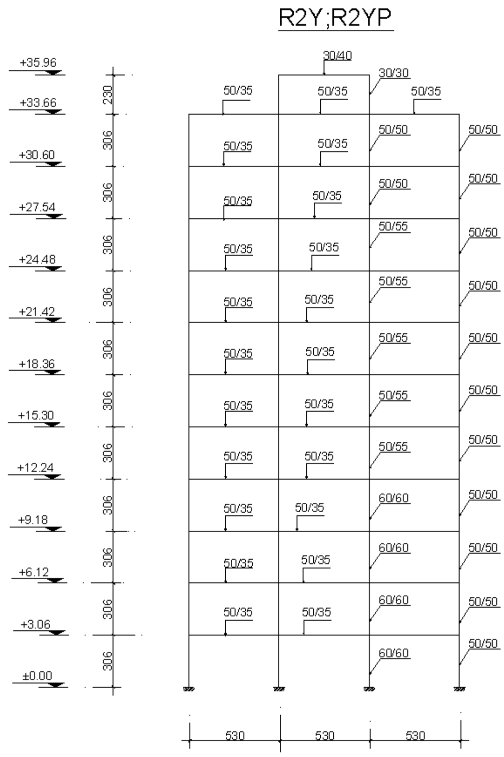
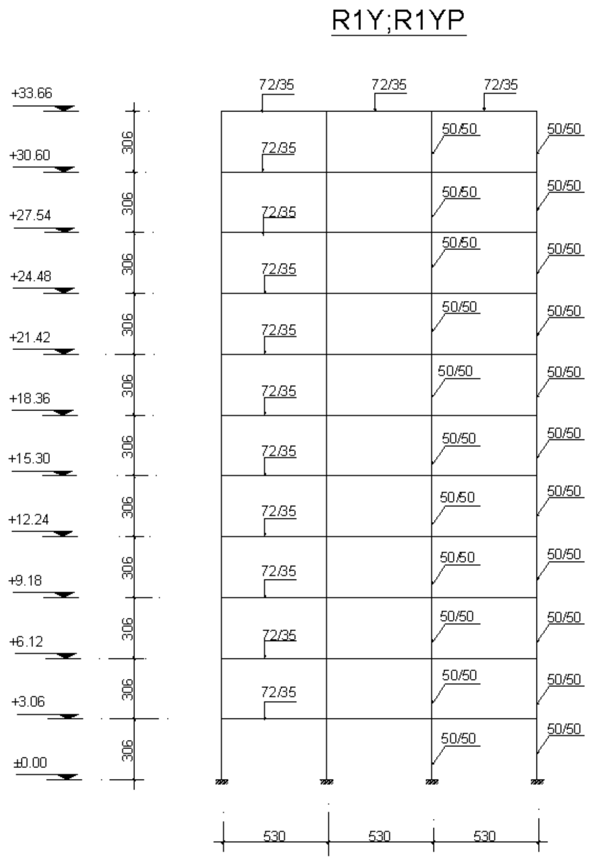


Figure 2 Type of Frames in X-X and Y-Y directions



Elastic and inertial characteristics of the structure

Inertial and elastic characteristics of the structure are necessary as input data file of TABS77 computer program.

Masses for each storey are shown in the following table taken from the original design performed by the authors of the project.

Table 1 Masses for each storey

Story	12	11	10	9	8	7	6	5	4	3	2	1
Mass	13.34	49.5	41.9	41.9	41.9	41.9	41.9	41.9	41.9	41.9	41.9	41.9

$$m_i = G_i / 98.1 \left[KN / (dm / s^2) \right] \quad (1)$$

The total seismic force acting on building, S, shall be determined according to formula:

$$S = K \times G \quad (2)$$

Where: K- is the total seismic coefficient for the horizontal direction and

G- is the total weight of the building and its equipment.

The total seismic coefficient K shall be calculated from the expression:

$$K = K_0 \times K_S \times K_d \times K_p \quad (3)$$

K_0 - the coefficient of the building category, K_0=1
K_S - the coefficient of the seismic intensity, K_S=0.1
K_d - the coefficient of dynamic response, K_d=0.6
K_p - the coefficient of ductility and damping, K_p=1

Seismic coefficient Ks=0.1 (Zone Seismicity IX on the MSC Scale) was adopted for calculation of lateral seismic forces according to Code of Technical Regulations for the Design and Construction of Building in Seismic Regions (Former S.F.R Yugoslavia).

The storey seismic force S_i is determined by distribution of the total ones according to approximate formula:

$$S_i = 0.85 \times S \times \frac{G_i \times H_i}{\sum_{i=1}^{n} G_i \times H_i} \tag{4}$$

For the top story, a part of above mentioned distributed value is assumed the remainder to act as concentrated load equal to 0.15 of S. The calculation of those forces is given in table 2.

For the consider structure and above forces, appropriate model in program TABS77 was made for obtaining static forces and deformations based on linear analysis. The dimensions of elements, mentioned above, were used.

Table 2 The storey seismic force S_i

Story	Gi (kN)	Hi (m)	Gi*Hi(KNm)	S_i0.85 (kN)	S_i0.15 (kN)	Si(kN)
12	1314.5	35.96	47269.42	126	426	552
11	4805.8	33.66	161763.23	434.4		434.4
10	4120.2	30.6	126078.12	337.2		337.2
9	4120.2	27.54	113470.31	303.6		303.6
8	4120.2	24.48	100862.5	270		270
7	4120.2	21.42	88254.69	236.4		236.4
6	4120.2	18.36	75646.88	202.2		202.2
5	4120.2	15.3	63039.06	168.6		168.6
4	4120.2	12.24	50431.25	135		135
3	4120.2	9.18	37823.44	101.4		101.4
2	4120.2	6.12	25215.62	67.2		67.2
1	4120.2	3.06	12607.82	33.6		33.6
			Σ= 902462.34			

Non-linear response of the structure

Non-linear analysis was performed by computer program NRES for seven earthquakes records (Petrovac, Ulcinj-Albatros, Ulcinj-Olimpik, Bar, Bitola94, El Centro and Parkfield) with PGA's from 0.02g to 0.55g (more than 150 non-linear analyses). Response spectra of those records are presented in the following diagram.

Shear-type-lumped mass model of the structure was adopted with the ultimate ductility 4 (according to YU codes for moment resistant frame structures), and ultimate ductility 8 for comparison.

Bilinear 10 % strain-hardening diagram was adopted for modelling of non-linear behaviour of elements.

Summarized results as elastic characteristics of considered structure are presented in table below:

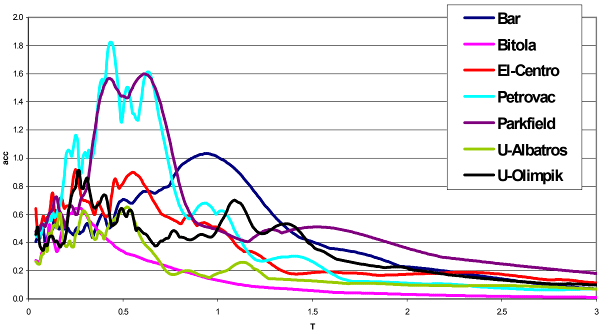


Figure 3 Response spectra for seven earthquakes records

Displacements Δ^* given in this table are displacements data used as input along with mass and storey stiffness used of NRES computer program to complete procedure for calculation of damage on the structure in the next step.

RESULTS AND DISCUSSION

Damage and vulnerability assessment

Final results of NRES computer program-capacity curve transformed in ADRS spectra complete the base for calculation of damage on the structure by means of DAMAGE computer program. DAMAGE calculates the value of damage index "DI" as a function of spectral displacement Sd at the top of the equivalent SDOF structure.

The modified Park and Ang's model is used as parameter for expression of damage. The modification of this model refers to deformations:

$$DI = \frac{Dm - Dy}{Du - Dy} + \beta_e \frac{\int dE}{Fy \cdot Du} \tag{5}$$

Fragility curves represent one of the possible forms of the *earthquake intensity-damage* to structures relationship. So, fragility curve shows probability that the damage under an earthquake of a given intensity will ***exceed*** a certain damage state:

$$P\left[DI \geq DI_K\right] = 1 - \phi\left[\frac{DI_K - DI}{\sigma}\right] \tag{6}$$

Table 3 The results of structural analysis

Storey	Displ.X-X (dm)	Δ_{XX} (cm)	Displ.Y-Y (dm)	Δ_{yy} (cm)	Qi (kN)	K_{xx} (kN/cm)	K_{yy} (KN/cm)	Δ_{XX}^* (cm)	Δ_{yy}^* (cm)	θ
12	0.646	0.42	0.722	0.49	552	1314.29	1126.53	0.4788	0.5586	0.1
11	0.604	0.34	0.673	0.35	986.4	2901.18	2818.29	0.3876	0.399	0.1
10	0.57	0.43	0.638	0.45	1323.6	3078.14	2941.33	0.4902	0.513	0.1
9	0.527	0.49	0.593	0.54	1627.2	3320.82	3013.33	0.5586	0.6156	0.1
8	0.478	0.55	0.539	0.6	1897.2	3449.45	3162	0.627	0.684	0.1
7	0.423	0.61	0.479	0.67	2133.6	3497.7	3184.48	0.6954	0.7638	0.1
6	0.362	0.64	0.412	0.72	2335.8	3649.69	3244.17	0.7296	0.8208	0.1
5	0.298	0.68	0.34	0.76	2504.4	3682.94	3295.26	0.7752	0.8664	0.1
4	0.23	0.67	0.264	0.78	2639.4	3939.4	3383.85	0.7638	0.8892	0.1
3	0.163	0.67	0.186	0.77	2740.8	4090.75	3559.48	0.7638	0.8778	0.1
2	0.096	0.61	0.109	0.7	2808	4603.28	4011.43	0.6954	0.798	0.1
1	0.035	0.35	0.039	0.39	2841.6	8118.86	7286.15	0.399	0.4446	0.1

Δ - Elastic relative storey displacement (cm)
Qs - Storey lateral force (kN)
K - Storey stiffness
θ -Strain –hardening ratio
Δ^*=1.14 Δ

For obtained values of damage index are given the respective damage states to the structure:

$DI \leq 0.1$ None
$0.1 < DI \leq 0.2$ Slight damage
$0.20 < DI \leq 0.40$ Moderate damage
$0.40 < DI \leq 1.0$ Extensive damage
$DI \geq 1.0$ Collapse damage

For any discrete damage state in the structure is obtained fragility curve:

$$P\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{S_{d,ds}}\right)\right] \tag{7}$$

The damage probability matrices are another form of the *earthquake intensity-damage* to structures relationship. So, the damage probability matrix gives the probability that a certain damage state will be ***achieved***:

$$P\left[D_S\right] = \frac{\phi\left(\frac{DI_{K+1} - DI}{\sigma}\right) - \phi\left(\frac{DI_K - DI}{\sigma}\right)}{\phi\left(\frac{DI_n - DI}{\sigma}\right) - \phi\left(\frac{DI_l - DI}{\sigma}\right)} \tag{8}$$

Two programs VULN and VULNM were used for those calculations:

- VULN for obtaining Fragility Curve and
- VULNM for Damage Probability Matrix.

Results of analysis are given in attached Excel diagrams

Set parameters providing Fragility Curve: Ultimate Ductility MU=4

Table 4 The parameters of providing Fragility Curve, MU=4

	PDs1	PDs2	PDs3	PDs4
E(Sd) cm	3.800	6.200	8.270	14.600
β	0.299	0.227	0.234	0.168

Table 5 The parameters of providing Fragility Curve, MU=8

	PDs1	PDs2	PDs3	PDs4
E(Sd)cm	5.95	10.11	13.44	25.7
β	0.299	0.227	0.234	0.168

E(Sd) cm -median value of spectral displacement
β - coefficient of variance dependant of the uncertainties in estimation of damage state.

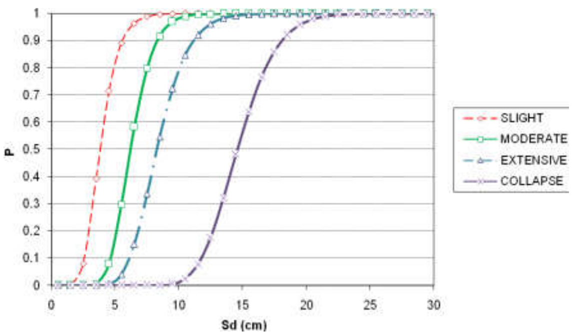


Figure 4 Fragility Curve (Ultimate Ductility MU=4)

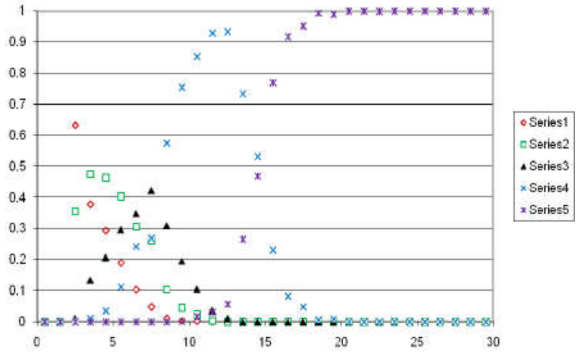


Figure 5 Damage Probability Matrix (Ultimate Ductility MU=4)

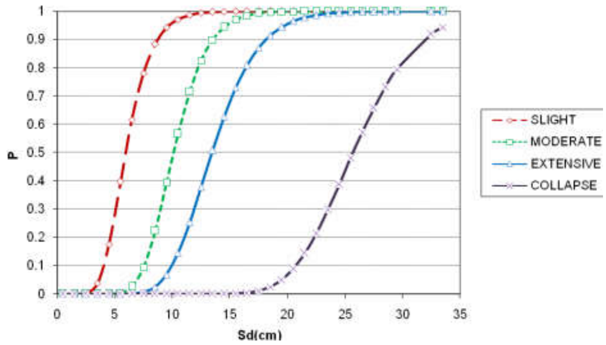


Figure 6 Fragility Curve (Ultimate Ductility MU=8)

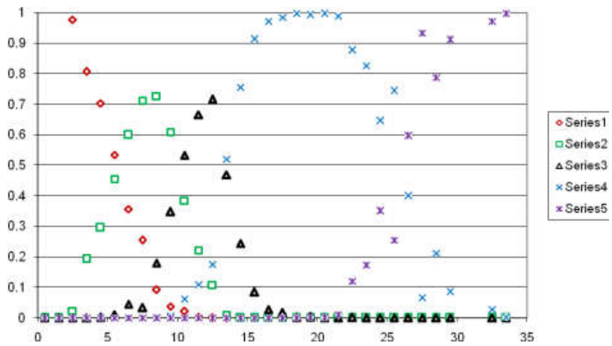


Figure 7 Damage Probability Matrix (Ultimate Ductility MU=8)

CONCLUSION

Structure considered in this paper, was design and performed according to YU codes. Up to the certain level of interstory drift each story behaves elastically. Values of the interstorey drifts have been calculated by using TABS77 computer program. Seismic forces at the floor levels were designed according to equivalent static method.

Ultimate ductility of 4 has to be provided in each vertical element as more realistic for moment resistant R/C frame structures.

Results of the performed analysis have shown that:

- For spectral displacements up to 2cm, no damage.
- For spectral displacement from 2cm to 4cm slight damage.
- For spectral displacement from 4cm to 6cm moderate damage.
- For spectral displacement from 6cm to 10cm extensive damage.

- For spectral displacement over 10cm exists considerable probability to complete collapse of the structure.

For a certain value of Spectral Displacement Sd for example Sd=10cm from Fragility curve it might point out the following:
100% exceedance probability of slight structural damage.
97% exceedance probability of moderate structural damage.
78% exceedance probability of extensive structural damage.
2% probability of complete collapse of the structure.

For of ***ultimate ductility of 8*** results of the performed analysis have shown that:

- For spectral displacements up to 3cm, no damage.
- For spectral displacement from 3cm to 5.5cm slight damage.
- For spectral displacement from 5.5cm to 7cm moderate damage.
- For spectral displacement over 7cm to 17.5cm extensive damage.
- For spectral displacement over 17.5cm exists considerable probability to complete collapse of the structure.

So it is obvious better structural behaviour for higher values of ultimate ductility. For a certain value of Spectral Displacement Sd, for example Sd=10cm from Fragility curve it might point out the following:

96% exceedance probability of slight structural damage.
48% exceedance probability of moderate structural damage.
11% exceedance probability of extensive structural damage.
No probability of complete collapse of the structure.

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