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DEVELOPMENT OF ANALYTICAL FRAGILITY AND VULNERABILITY FUNCTIONS OF LARGE-PANEL RESIDENTIAL BUILDINGS TYPICAL OF BULGARIA

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Abstract: *During the 1960 to 1990 period many countries across the Eastern Europe and Central Asia addressed to shortages in housing by the mass production of precast large-panel multi-family buildings (LPBs). These structures are currently the home of millions of people in regions with medium-to-large seismicity and yet, the seismic risk associated with them is not well understood. A key gap in a consistent risk analysis process is the absence of fragility and vulnerability functions developed specifically to capture the peculiarities of the LPBs. This issue is the scope of the presented study that is focused on the LPBs in Bulgaria. New damage criteria for LPBs were defined in this study and used to differentiate the damage states. Several structural architypes were defined based on similarity of key structural parameters and then, 3D nonlinear FE models were built, and their complex failure patters were analysed. The influence of building height, aspect ratio in plan, degradation of material strength and post-construction alterations were also investigated. Nonlinear static analyses were used to derive the EDPs needed to define the mean seismic capacities of the fragility functions for different damage levels. The vulnerability functions were obtained using custom consequence models. The outcome is a dataset of structural fragility/vulnerability functions for the LPBs typical of Bulgaria, which could be extended to cover similar typologies in other countries. This is a substantial improvement on the existing database which is currently lacking analytical fragility and vulnerability function explicitly for LPBs.*

Introduction

Large-panel buildings (LPBs) are one of the most common types of residential buildings in Bulgaria and in the broader Europe and Central Asia region (ECA). Built primarily during the rapid urbanisation of the 1960-1970’s these buildings were assembled by joining together of pre-cast RC panels allowing rapid mass construction. The panels are connected only in discrete locations through grouted dowels resulting discontinuity of the lateral resistant system. Therefore, the LPBs have very different seismic response compared to classical RC shear wall buildings with similar wall configuration. In addition, the seismic response of these buildings is one of the least studied analytically, partially because of the complexity for numerical modelling.

Many of these buildings are in seismic prone regions and the result is that there are millions of people that are living in buildings for which the magnitude of the associate seismic risk and the socio-economic impact of earthquakes with different intensities is not well understood, neither by the decision makers, nor by the practicing engineers. In the same time large-panel buildings are unavoidable element in the housing strategy of almost every country in ECA. On average they form about 20-30% from the residential building stock in these countries and most of them approached or exceeded the design “life time”. The issue is addressed differently from country to country (or even within a single country) with solutions spanning between the two extremes – from demolishing and replacement with new building to complete rehabilitation and extension with additional floors. In seismic prone countries, the decision making needs to consider also the associated largely unqualified seismic risk.

The topic attracted the attention of the World Bank which, with funding from Global Facility for Disaster Reduction and Recovery (GFDRR) initiated a study on the topic through the project

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"Probabilistic Seismic Risk Assessment and Seismic Safety Improvement Recommendations for pre-1990 Multi-Family Apartment Buildings in Bulgaria and broader Europe Central Asia region". One of the main challenges in the project was to bridge the gap in the global earthquake engineering knowledge database that lacked analytical fragility and vulnerability functions specifically derived for large-panel buildings. The development of these functions is the main topic of this paper and to our knowledge the fragility and vulnerability functions presented herein are the first analytical ones for large-panel buildings published in the international scientific literature.

In the current paper only the development of region- and LPB-specific fragility functions, consequence models and vulnerability functions for structural damage and loss is discussed.

Damage limit states

The definition of the damage states used in this study was based on the homogenised reinforced concrete (HRC) scale of Rosetto and Elnashai (2003). The description of the damage associated with each damage state (DS) was modified to consider the particularities of large-panel buildings, namely discontinuity of the load-bearing system and discrete connections of the panels through dowels. The seven damage states of the HRC scale were reduced to six damage states with "Partial Collapse" and "Collapse" merged into one damage state. The description of the expected performance in terms of habitability, see Table 1, is based on ASCE 41-13 and the risk for injuries/fatalities at each damage state is based on FEMA P-58-1.

Damage State	**Direct structural damage**
DS0 No damage	None
DS1 Slight damage	Fine cracks on the plaster of panels, especially around openings, on the interface between dowels and panels. Micro cracks in the dowels.
DS2 Light damage	Onset of structural damage, diagonal hairline cracks in structural panels and on the interfaces between panels. Dowels are easily identified due to the cracks around and diagonally through the grouting
DS3 Moderate damage	Dowel infill concrete crashed, reinforcement in the dowel may have yield. Diagonal cracks in internal panels are formed.
DS4 Extensive damage	Dowels are completely damaged. Panels have experienced significant cracking and noticeable residual displacement
DS5 Collapse	Complete or partial building collapse

Table 1. Definition of DSs for LPBs, after Rosetto and Elnashai (2003) and ASCE 41-13

Large-panel buildings have a seismic response different from conventional RC shear wall structures, due to the discontinuity of their lateral load-resisting system which consists of structural elements connected via localised connections in the form of grouted in-situ dowels (or welded steel plates in some countries in Central Asia). Damage in the dowels and in the panels need to be assessed separately, since each of these components can trigger global failure. However, an accurate estimate of the direct financial loss require combination of these two damage modes. Storey drifts are used as the main Engineering demand parameter (EDP) to monitor/control the exceedance of a Limit State (LS), i.e. the transition between damage states (DS). The total storey drift in a large-panel building is a combination of the relative displacement in the dowels and in the panels. The two drifts are monitored separately, and the maximum equivalent storey drift of each component was used to trigger the exceedance of a limit state.

The total dowel and total panel damage level at each floor *i* was assed via damage indexes (DI). For each storey, two DIs were calculated, $DI_{D,i}$ for all dowels on floor i and $DI_{P,i}$ for the panels on floor *i*, respectively. The damage index for all dowels/panels on floor i was calculated as follows:

$$DI_{D/P,i}=\frac{\Delta_{D/Pr,i}-\Delta_{D/Py}}{\Delta_{D/Pu}-\Delta_{D/Py}} \tag{1}$$

where: $DI_{D/P,i}$ – DI for the dowels/panels for storey *i*; $\Delta_{D/Pr,i}$ – drift at the response recording for storey *i* considering only the drift in the dowels/panels; $\Delta_{D/Py}$ – drift at "yielding" considering only the drift in the dowels/panels (corresponds to DS1 in Table 2); $\Delta_{D/Pu}$ – drift at "collapse" considering only the drift in the dowels/panels (corresponds to DS5 in Table 2).

The composite damage index (CDI) for each storey was calculated as sum of the two DIs using weight factors W_P and W_D. Since the panels are more expensive to repair, their DI were given priority with W_P=0.67 and W_D=0.33. The CDIs for the entire building, used to corelate the damage with the repair cost, were derived as weighted average of the CDIs of each floor *j*, weighted with the gross wall area A_j at each floor. The drift limits that trigger the exceedance of the limit states for the dowels were based on the experimental results of Orlinov (2015), also discussed below. The drift limits for the panels were based on FEMA P-58-1.

Limit State	**Drift ratios**		**CDI**
	Dowels, after Orlinov (2015)	**Panels, after FEMA P-58-1**	
LS1	0.02%	0.07%	0.01
LS2	0.05%	0.55%	0.17
LS3	1.07%	1.09%	0.43
LS4	2.50%	1.30%	0.59
LS5	5.00%	2.00%	1.00

Table 2. Definition of Damage Indices and the Composite Damage Index.

Finite element modeling

Structural modelling approach

The main features of the large-panel structures are their modular regularity in plan and the cast-in-place, vertical and horizontal, joints between the precast panel elements. Dowels must be adequately modelled in FE analyses, so the initial strength and its degradation, the load paths and the governing global failure mechanisms are properly simulated. Example of a detailed FE model of a large-panel building is provided on the left side in Figure 1.

Significant efforts were devoted to testing, calibration and validation of different options (material models, element formulation and mesh size) for adequate modelling of the dowels and the interaction surfaces between the panels. The results from a recent experimental campaign on dowels completed in 2015 in UACEG, Bulgaria by Orlinov (2015) were selected as representative. Several scaled specimens of dowels typical of the Bulgarian nomenclatures of large-panel residential buildings were tested under cyclic static shear force with different clamping (axial) forces. The same specimens were modelled in LS-DYNA and analysed under monotonic or cycling loads. View of the micro-models and validation of numerical results with experiments for two specimens with different axial loads are shown on the right in Figure 1.

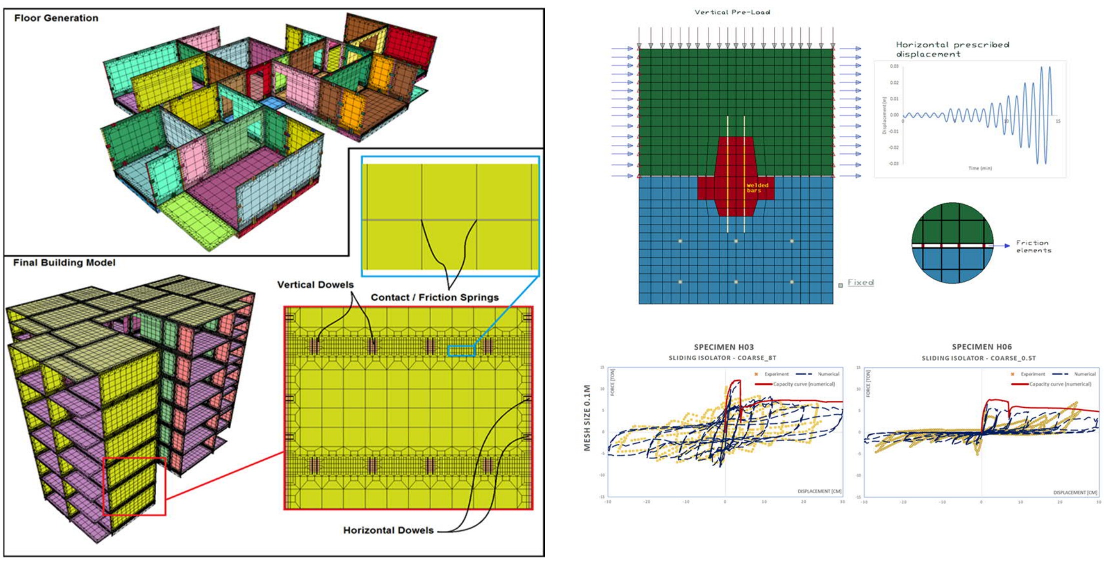

Figure 1: Building the FE model from the predefined library of FE models of panels (upper left corner), a complete 3D model with detail of the dowel modelling (lower left corner), micro FE models used to calibra2te the dowel models (upper right corner) and numerical versus experimental results from the cyclic and push-over analysis of two specimens.

Description of the finite element models

Since, it was not feasible to model explicitly each structural type out of more than 100, a reasonable number of structures was selected for direct non-linear analysis based on the similarities in their structural characteristics. The full-scale models of nine structural architypes were built with an automated procedure, considering the conclusions reached after the analysed micro-models and the performed parametric studies. The baseline models were further modified to consider variation in number of floors, number of sections. In addition, modifications of the models were used for a sensitivity study of the effect of accelerated corrosion in dowel rebars and of unauthorised structural interventions (openings for doors) in the panels at the lower floors.

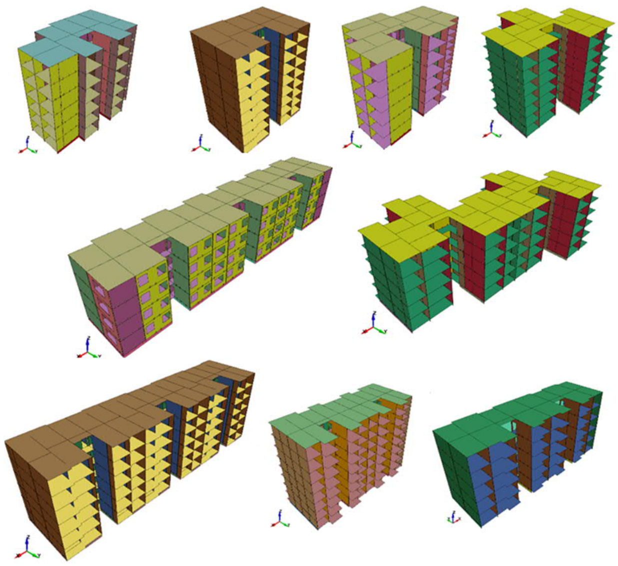

Figure 2: 3D view of the FE models of the nine LPB architypes (mesh not shown).

Definition of capacity curves

General approach

The structural capacity curves (diagrams), used later for the calculation of the analytical fragility functions, were derived from the force-displacement curves obtained from the FE analysis together with the defined damage limits in terms of drifts for the dowels and the panels separately. The graphs on the left side in Figure 3 illustrate the process, using as an example the most common typology in Bulgaria, “Classics_8F_1S” having 8 floors and 1 section in X direction. The storey drifts at each floor are monitored considering the relative displacements of dowels and panels separately. The first exceedance of a certain damage limit, either for a panel or for a dowel, marks the pseudo-time at which the displacement profile is used to calculate the equivalent SDOF system for the corresponding damage state. The process is repeated for each damage state and the responses of all SDOFs are combined in one capacity curve plot in ADRS format. The capacity curves in longitudinal (X) and transverse (Y) direction together with the deformed shapes of the models at each DS for typology “Classics_8F_1S” are plotted on the right side in Figure 3.

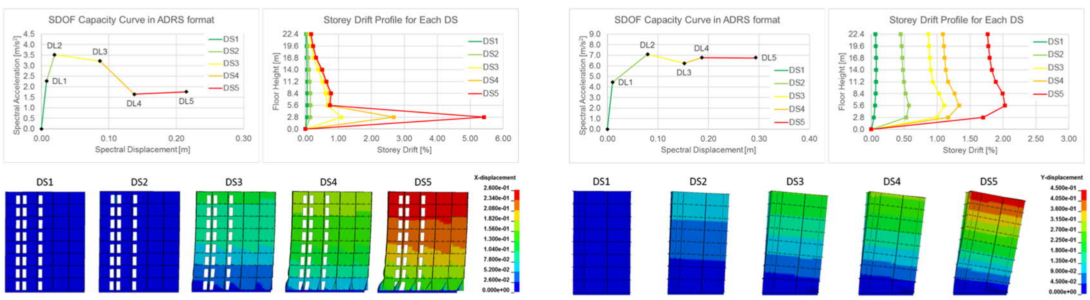

Figure 3: Derivation of simplified capacity curve (left) and example of capacity curves, storey drift profiles and deformed shapes for the most common LPB structural architype (right).

Fragility functions for structural damage

Estimation of the median capacities for structural damage

The median capacity for each damage state was estimated using the capacity curves derived from the non-linear FE models. The FEMA 440 Capacity Spectrum Method (CSM) was adopted with slight modifications considering typical modes of failure of the multi-family LPBs.

The procedure includes the following steps, repeated for each horizontal direction of seismic input: 1) Calculate the ADRS (acceleration-displacement response spectrum) for the representative UHRS (with shape averaged across Bulgarian territory), normalized to A_m=1.0g. 2) Develop the capacity diagram for the structure up to collapse (DS5), approximated with straight lines between DSs. The entering points of the DSs will be the performance points. 3) Transform the capacity diagram for equivalent bi-linear SDOF with idealized yielding point. For each damage state i, calculate the ductility ratio $\mu_i=d_i/d_y$. For DS1, μ_1 is set to 1.0. 4) Using the calculated, μ_i, calculate effective viscous damping $\beta_{eff,i}$ and effective period $T_{eff,I}$. 5) Using the effective damping (Step 5), adjust the initial ADRS demand for $\beta_{eff,i}$. 6) Calculate the modified ADRS (MADRS) accelerations. An additional scaling factor K_{pmc}<2.0 was added to extend procedure for stable response after reaching maximum capacity. 7) Determine the MADRS displacements (improved displacement modification procedure). 8) Scale the median capacity appropriately, so the MADRS intersects the capacity diagram at the performance point for DSi. The scaling factor is the median capacity $A_{m,i}$ in g.

The CSM solution is summarized in Figure 4 for “Classics_8F_1S”, DS3 in the weaker X-direction. The calculated median capacity for this typology and direction is $A_{m,DS3}$=0.77g.

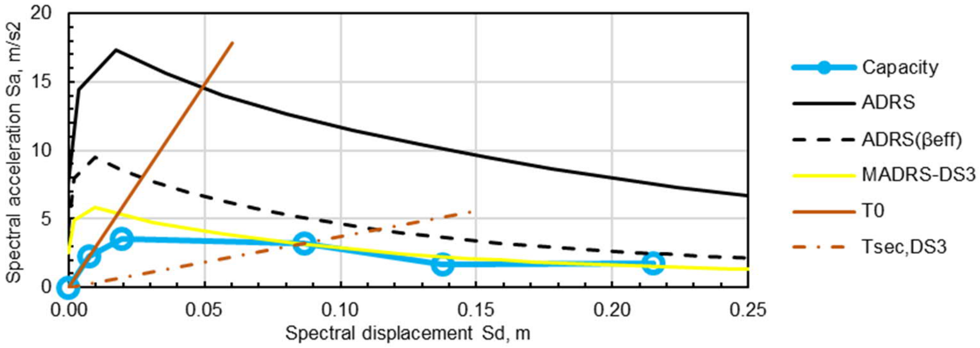

Figure 4. CSM solution for “Classics_8F_1S” in X direction, DS3

Estimation of variability and uncertainty of analytical fragility functions for structural damage

The uncertainty in the fragility functions was adopted from the recommendations by FEMA P-58-1. The composite log-standard deviation β include components of apparent randomness β_R and model uncertainty β_U. In Table 3 the uncertainty values are given for both “nominal” and “reduced” (deteriorated materials, interventions in load-bearing walls, etc.) structural properties. The fragilities for typology “Classics_8F_1S” in X direction for all DSs are plotted in Figure 5

Uncertainty component	**DS1,2 nominal**	**DS3,4,5 nominal**	**DS1,2 reduced**	**DS3,4,5 reduced**
Material strength (R)	0.12	0.12	0.12	0.12
Construction quality (R)	0.10	0.10	0.10	0.10
Building definition and construction QA (U)	0.25	0.25	0.40	0.40
Analytical model quality & completeness (U)	0.25	0.25	0.25	0.25
Ground motion variability (R)	0.00	0.00	0.00	0.00
Record-to-record variability for drift (R)	0.20	0.40	0.20	0.40
Modelling of seismic demand (U)	0.20	0.20	0.20	0.20
Total (C) – SRSS of all above	0.48	0.59	0.57	0.67

Table 3. Uncertainties in the analytical fragility functions for structural damage, adopted from FEMA P-58-1 with modifications

Spatial combination of fragility functions

The Damage States were defined on a building scale to be consistent with the definition of the hazard (geometric mean) and the arbitrary orientation of buildings in each asset. Both X and Y direction damage modes contribute to the conditional probability of the building being in each Damage State, and the damage modes are not mutually exclusive. A common cause adjustment was performed using the "theorem of unimodal limits" as suggested in the SPANCOLD (2012) guideline for risk assessment of dams. The upper limit of the combined probability was assumed such that the damage in one direction of the building does not prevent the damage in the other direction. the fragility functions in X and Y directions have the same deviation $\beta_x=\beta_y$, and the combined fragilities fit very well to log-normal distributions with parameters A'_m and β'. The combined structural fragilities for "Classics_8F_1S" are plotted in Figure 6 (dashed lines show the maximum of the X and Y fragilities, and the solid lines are the combined fragilities).

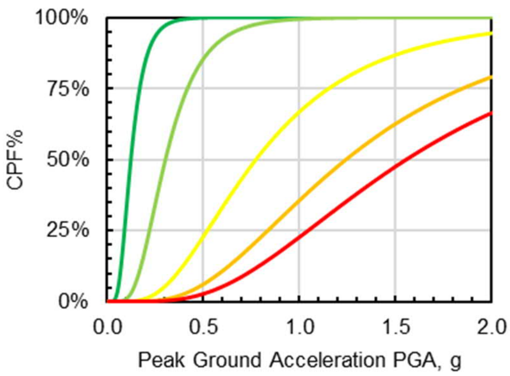

Figure 5. Structural fragilities for "Classics_8F_1S" in X direction

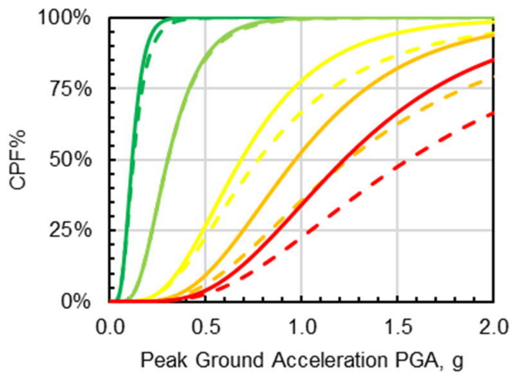

Figure 6. Combined structural fragilities for "Classics_8F_1S"

Summary of structural fragilities for large-panel building typologies in Bulgaria

In this risk study 11 out of more than 100 structural typologies were explicitly modelled and analysed for their strength and displacement capacity to produce fragilities. For the two most common typologies, additional analyses were run with "reduced structural properties". The fragility functions for the other typologies were derived based on the analytical ones by scaling the median capacities and keeping the log-standard deviation of the original fragilities. The scaling coefficients were obtained from the correlations between the basic fragilities for structures with different number of stories and different number of sections. The median capacities A'_m and the log-standard deviations β' of the fitted combined fragility functions for the analysed typologies are listed in Table 4.

Structural types	**DS1**		**DS2**		**DS3**		**DS4**		**DS5**	
	A_m'	**β'**	**A_m'**	**β'**	**A_m'**	**β'**	**A_m'**	**β'**	**A_m'**	**β'**
Classics_8F_1S	0.11	0.45	0.30	0.45	0.68	0.50	0.96	0.50	1.21	0.50
Classics_6F_1S	0.15	0.43	0.30	0.43	1.04	0.50	1.18	0.50	1.53	0.50
Classics_6F_3S	0.16	0.43	0.31	0.43	1.01	0.52	1.17	0.52	1.56	0.52
Tetris_7F_1S	0.15	0.41	0.43	0.41	0.87	0.50	0.98	0.50	1.41	0.50
Tetris_5F_1S	0.18	0.41	0.39	0.41	0.96	0.50	1.08	0.50	1.73	0.50
Tetris_5F_2S	0.22	0.40	0.36	0.40	0.95	0.50	1.53	0.50	2.13	0.50
Pioneer_4F_3S	0.44	0.40	0.82	0.40	1.24	0.52	1.65	0.52	2.00	0.52
Confidence_8F_2S	0.15	0.45	0.23	0.45	0.75	0.50	0.94	0.50	1.25	0.50
Modern_6F_1S	0.19	0.41	0.44	0.41	1.42	0.50	1.64	0.50	1.94	0.50
Neoclassical_5F_2S	0.22	0.42	0.46	0.42	1.11	0.50	1.38	0.50	1.92	0.50
KingsValley_6F_1S	0.18	0.42	0.47	0.42	1.19	0.50	1.27	0.50	1.58	0.50
Classics_8F_1S_degr	0.11	0.53	0.21	0.53	0.76	0.57	0.93	0.57	1.21	0.57
Classics_8F_1S_degr	0.09	0.53	0.23	0.53	0.73	0.60	0.84	0.60	1.00	0.60
Tetris_7F_1S_open	0.13	0.50	0.32	0.50	0.87	0.57	1.08	0.57	1.49	0.57

Table 4. Parameters of fitted and combined analytical fragility functions for structural damage

Verification with data from past earthquakes

To verify the analytical fragility functions, the later were compared to empirical fragility curves that we derived based on the earthquake experience data after the 1988 Spitak Earthquake in Armenia (M6.8), the 1977 Vrancea Earthquake in Romania (M7.2), and the 2012 Pernik Earthquake in Bulgaria (M5.6). Due to the limited information on the damage to large-panel buildings and particularly due to the lack of information for moderate and extensive damage levels, the comparison of the analytical and empirical fragility functions was performed only for the lower damage states – slight to light damage defined as DS1 and DS2 in this study.

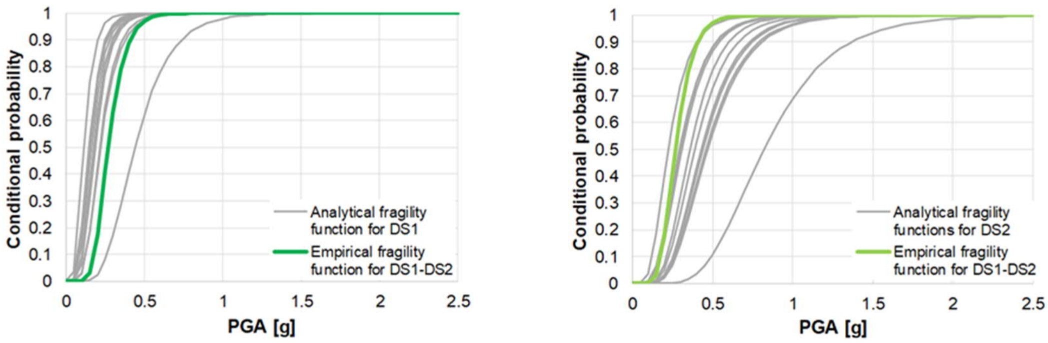

Figure 7. Comparison of the analytical fragility functions for DS1 and DS2 for all analysed typologies with empirical fragility functions for DS1 and DS2

Consequence models

Development of consequence models for structural loss

The steps for the development of the Consequence Models for structural damage are listed in Figure 8. In this figure RRC means “Repair or replacement costs” and LR means “Loss ratio”.

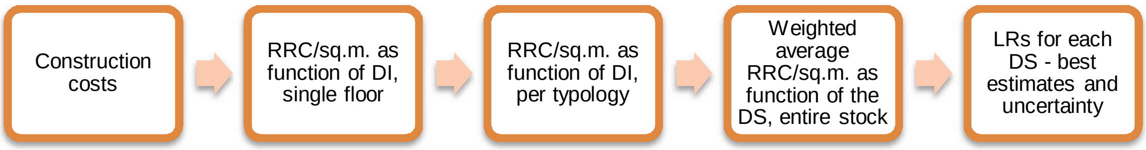

Figure 8. Development of Consequences Models for structural damage

The construction costs were estimated based on statistics about the market price in Bulgaria for 2018, collected from the databases of local real estate agencies. The market prices across the country were lumped in three groups (towns; cities; three biggest cities) and construction cost was set at approximately 50% of the market price with expected price surge included.

The repair costs for a single storey were calculated from the CDI defined previously. The repair costs were estimated based on the current cost of construction and repair works in Bulgaria. They were calculated for the CDI corresponding to entering each DS (from 1 to 5). DS4 and DS5 were associated with irreparable damage and thus with the replacement cost.

The repair costs are a function of the DI for each storey. It was assumed that in a multi-storey building a DS is reached when the maximum DI along the height exceeds a given threshold, and the DIs for the other storeys are less than the maximum. To reflect the expected damage distribution throughout a building, the repair costs for DS1 to DS3 for each storey of each analysed typology in both directions were calculated, and their average was taken as the representative repair cost per sq.m. for the typology. For DS4 and DS5 the replacement cost was used for the entire buildings.

One representative cost for all typologies was estimated, by averaging the cost among the analysed typologies. For each DS, mean value and uncertainty of the repair cost were calculated.

An example for the costs in Sofia is plotted in Figure 9: single storey in top left, entire building for different typologies in top right; and entire building averaged for all typologies in the bottom.

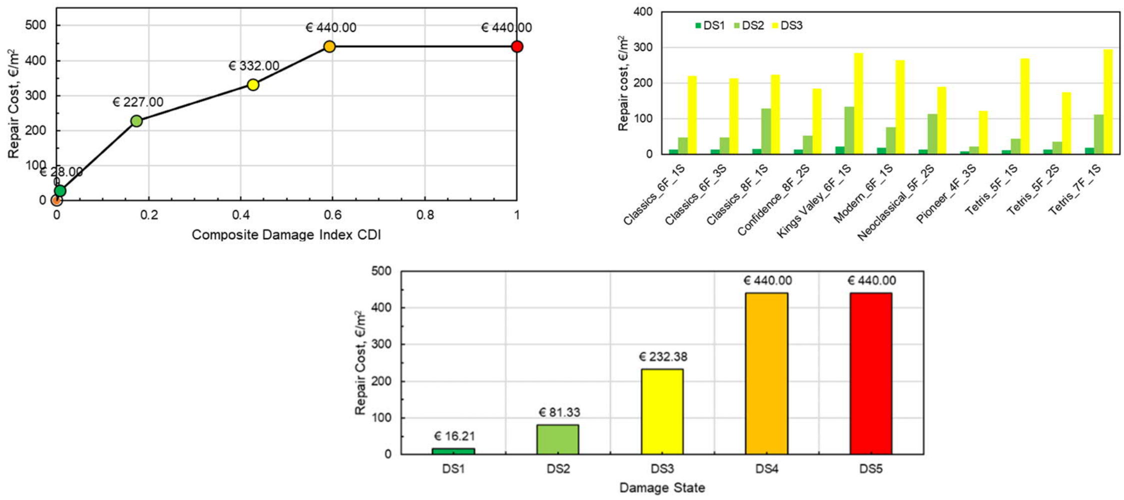

Figure 9. Repair/Replacement costs for Sofia, per square metre (top left – single storey; top right – entire building, all typologies, bottom – entire building, averaged)

Consequence model for direct structural loss

For each Damage State *i* the mean Loss Ratio LR_i is calculated as the quotient of the Representative Repair Cost RC_i and the Construction cost for a new building, set at €473.00 per square metre for Sofia. The Consequence Model for structural damage with Mean and Coefficient of variation (CoV) of the LRs is shown in Table 5.

DS	**Damage level**	**Replacement cost ratio**	
		Mean	**CoV**
DS1	Non to slight structural damage	0.032	20%
DS2	Light structural damage	0.162	28%
DS3	Moderate structural damage	0.467	14%
DS4	Extensive structural damage	0.914	2.4%
DS5	Complete structural damage, collapse	0.926	2.7%

Table 5: Consequences Model for direct loss due to structural damage

Vulnerability functions

Vulnerability functions for direct financial loss due to structural damage

The vulnerability functions used to perform the risk analyses were evaluated from the associated fragility and consequence models to the respective asset/taxonomy/component.

For each of the Damage States defined in the fragility model, the expected value (mean) of the Loss Ratio (LR) was estimated in the Consequence model. Since the Loss Ratio may vary the coefficients of variation were provided for reach Loss Ratio. The loss curve was then derived by obtaining the Loss Exceedance Ratios (LERs) at different Intensity Measure levels. The absolute losses were then estimated from the product between the Loss Exceedance Ratios and the cost values for the asset from the exposure model. The fragility functions were used for the damage distribution analysis and the vulnerability functions were used for the risk analysis. In Figure 10 sample Vulnerability Function for direct financial loss due to structural damage is shown. All of them use the fragility functions for typology Classics_8F_1S.

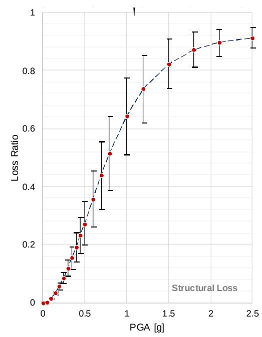

Figure 10. Sample vulnerability Function for direct loss due to structural damage

Discussion

Large-panel buildings and the other types of residential buildings in Bulgaria

Figure 11 shows a comparison between the fragility functions of several large-panel structural typologies and selected fragility functions for a few other structural typologies that are common for Bulgaria. The fragility functions on the left are for light damage (onset of monetary loss) and the those on the right are for collapse. The fragility functions for RC Dual Buildings, Mid-rise RC frame with masonry infills, Shear wall buildings (pre-code) and Mid-rise URM buildings are selected from the database of Syner-G (2014) research project. Evidently, the onset of structural damage is expected at similar seismic intensity, but the ultimate seismic capacity of large-panel buildings is significantly higher compared to most of the other older buildings with a probability of collapse under a “design” earthquake an order of magnitude lower. This important observation is also confirmed from the observed damage in past earthquakes in Romania and Armenia.

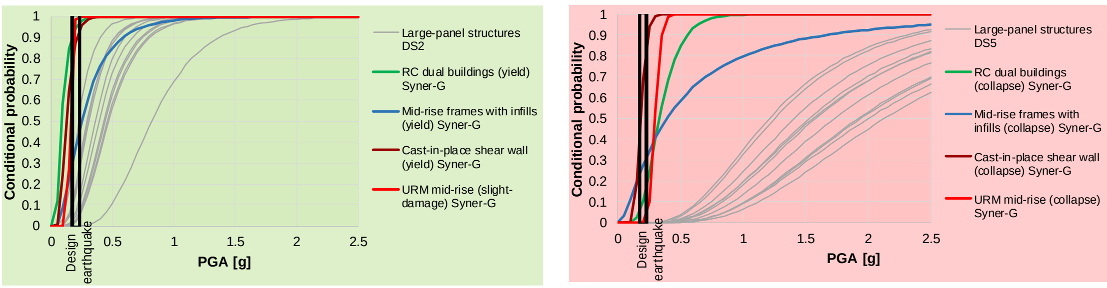

Figure 11: Comparison of the fragility functions for large-panel structural typologies for light damage (left) and collapse (right) with selected fragility functions for other structural types

Applicability of results to other countries in Europe and Central Asia

The fragility and vulnerability functions presented in this paper fill in a significant gap in the global earthquake engineering knowledge database. Although the reported fragility functions were developed for large-panel buildings typical of Bulgaria and with consideration for the local seismic hazard, the fragility functions in this report are believed to be the most accurate substitute for the fragility functions of similar LPBs in other countries. However, when using the presented fragility functions in other countries, the following aspects need to be considered: 1) Key structural characteristics and plan arrangement of the studied buildings and the reference buildings (for the adopted fragility function) need to be compared and structural similarities need to be proven; 2) The number of dowels per floor and the basic dowel characteristics (grouted dowels and/or welded plates) should be very similar and 3) the response spectrum shape representing the seismic hazard should be similar, otherwise the procedure to derive the mean capacities from the capacity curve needs to be repeated. As to the vulnerability functions, the Method of Repair for different damage and the repair costs need to be adjusted to the construction market conditions in the host country.

Further research and areas for improvement

Although the current fragility functions were derived based on detailed numerical models and nonlinear analyses, there are several areas for further research that would improve the understanding of the seismic response of large-panel buildings and would increase the credibility of the results. The most important that may have direct influence on the risk metrics are: 1) Modelling the SSI effects and assessing their influence on the seismic response. It is well known that SSI has a significant influence on the seismic response of rigid structures in soft soils and may have either beneficial or detrimental effects ; 2) Using nonlinear dynamic analysis would also improve the understanding of the seismic response and would quantify better the effects of hysteresis and friction damping, shift of the fundamental mode due to softening, cyclic degradation, duration of the strong pulses, record-to-record variability, etc.; 3) Using regional seismic hazard data will likely allow further differentiation of the fragility functions for different large-panel typologies according to their location.

Conclusion

The study presented in this paper is a pioneering work in several aspects. To our knowledge and based on the publicly available information, this is the first study that uses high-fidelity 3D non-linear FE models to analyse the seismic response of typical large-panel buildings under extreme seismic loads. The numerical models developed for this project consider explicitly all major factors that affect the seismic response of large-panel buildings far into the inelastic range – nonlinear material models for the panels, detailed modelling of the joints, modelling the discontinuity between the panels and the friction at their interfaces after the cracking of dowels. The most critical element of the model, the dowels, was validated by comparing their shear response under increasing cyclic loadings with experimental results. The level of detail of the numerical models was fundamental for the understanding of the global seismic response of large-panel buildings (sliding-friction energy dissipation mechanism) and to explain their favourable performance in past earthquakes. Another achievement in this project is the gained understanding of the major differences in the seismic response and seismic capacity among various large-panel building types. So far, in previous studies, all large-panel buildings were grouped in one or two types represented by one set of fragility functions. The fragility functions developed in this project are the other major contribution to the global earthquake engineering knowledge database. As far as we know, the fragility functions presented in this paper are the first ever analytical fragility functions for large-panel buildings. This study is also the first one that uses consequence models for seismic induced structural loss for large-panel buildings that are derived based on realistic methods for repair, specifically developed and priced for post-earthquake repair of different levels of damage to large-panel buildings in Bulgaria

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