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SEISMIC FRAGILITY OF WIDE-BEAM INFILL-JOIST BLOCK REINFORCED CONCRETE FRAME BUILDINGS

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Abstract

Wide-beam infill-joist block (WBIJB) reinforced concrete (RC) frame is a popular construction technique in Turkey and the Mediterranean basin. Even though there were previous observations and considerations about the problematic seismic behavior of the WBIJB frames, the behavior was concealed behind the general weaknesses of the RC frames. Currently WBIJB RC frames are not considered as a separate sub-class of RC frame systems during regional damage and loss estimation studies. However, the recent earthquakes shed a light on the possible inferior behavior of this class of building structures. The main difference of the WBIJB frames from conventional moment resisting frames is the presence of wide-shallow beams as an architectural preference. Due to shallow beams, the lateral stiffness of these systems decrease which resulted an increase in the period of the buildings. Hence, increased earthquake drift demands resulted. The main goal of the presented study is to develop the fragility curves for this specific class of RC frame buildings, which constitute a significant portion of the existing building stock in Turkey. The resulting fragility information could be employed in future regional loss estimation studies. For this purpose, a generic two dimensional frame is designed and modeled by considering the properties of the existing WBIJB frame buildings, the current seismic regulations and the construction practice in Turkey. Pushover and nonlinear time history analyses are conducted in order to quantify seismic demand and capacity. Then this information is used to calculate the probabilities of exceeding the predefined limit states as a function of seismic ground motion intensity parameters. In the last part of the study, the estimated fragility curves are compared with the available fragility curves of similar construction types.

Keywords: Wide-Beams; Beam-Column Connections; Fragility Curves; Reinforced Concrete.

1. Introduction

Wide-beam infill-joist block (WBIJB) RC frames are a variety of wide-beam systems where the floor slab is consisted of one way joists. Typically, the depth of the joists are same as the beams and space between the joists are spanned by a thin slab. In Turkish practice, the volume left under the thin slab, between the joists, are traditionally filled with light clay brick blocks or lately with styrofoam blocks (Fig. 1 and 2).

Popularity of reinforced concrete frames with WBIJB arises mainly from the architectural flexibility it provides and also the construction savings in the floor slab construction. Structurally, use of wide-beams could lead to smaller column sizes which still ensure the strong-column weak-beam requirements of the seismic codes due to the weaker beams. Therefore, relatively weak beams could develop failure mechanisms that could cause consumption of high amount of energy in a seismic scenario. Unfortunately, wide-beam frames are prone to systemic and element level weaknesses that could cause failures under earthquake demands.

The problems at the element level are due to the width and the shallow depth of the beams. Because of the larger width, longitudinal reinforcements of the beams that are left outside the core area of the columns have force transfer issues. Both the longitudinal column and beams bars have slip problems due to the limited anchorage lengths [1, 2, and 3]. The element level (specifically the beam-column joint) problems could be mitigated by enforcements of defined limitations on the sizes and the detailing requirements [4].

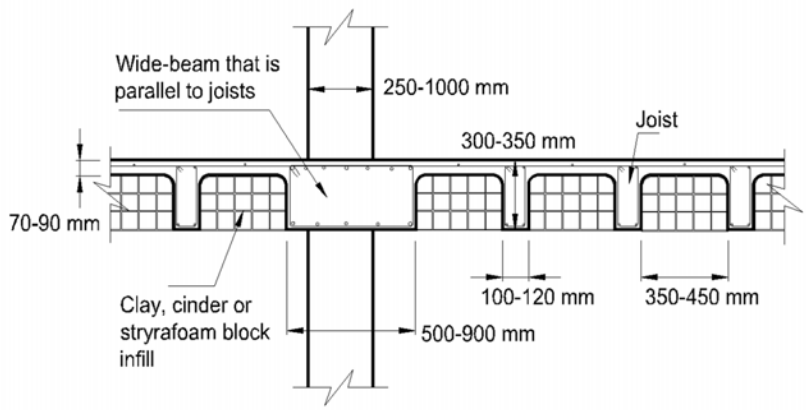

Fig. 1– Typical dimensions of Wide-Beam Infill-Joist Block Slab RC Frames in Turkey

In addition to issues listed above, there is a specific local problem to the applications in Turkey. Typical WBIJB frame construction in Turkey contains many eccentric beam-column joints. Currently, there is no known study about the behavior of eccentric wide-beam column connections.

The problems at the system level are the result of the decreasing stiffness of the moment frames due to the wide-beams. In a previous study [5], which concentrated on wide-beam RC frames in Turkey, it is shown that the average natural periods of studied structures increase about 40% (about a decrease of 50% in lateral stiffness). It should be noted that the increase in the period and the seismic displacement demand could be accepted to have the same order of magnitude.

It is known that wide-beam-column systems have inherently higher displacement capacities, but it is not clear whether the increase in the capacity is sufficient to match the increase in the demand. The field observations during the 2011 Van-Ercis, Turkey earthquake (M_w=7.2) indicates that WBIJB frames could have premature failures. A following study by Dönmez [5] confirmed that there are cases where the seismic displacement demand is larger than the capacity of the WBIJB frames. Therefore, further research is needed on the global response of the wide-beam RC frames.

Fig. 2– Wide-Beam Infill-Joist Block Slab Application in Turkey

The main goal of the presented study is to develop the fragility curves for this specific class of RC frame buildings. WBIJB RC frames constitute a significant portion of the total building stock in Turkey. Even though documentation is not available about the existing numbers, it is guesstimated that in general, the WBIJB frames are about 10-15% of the existing RC building inventory. In some local areas, this ratio goes up to 70%. Therefore, the resulting fragility information can be employed in future regional loss estimation studies. For this purpose, a generic two dimensional WBIJB RC frame is designed and modeled by considering the properties of the existing WBIJB frame buildings and the current seismic regulations in Turkey. Pushover and nonlinear time history analyses are conducted in order to quantify seismic demand and capacity. The numerical models of the frames are formed by use of the nonlinear springs that are lumped at the end of the members. The nonlinear springs are calibrated using the existing tests on wide-beam column connections. The analysis results are used to calculate the probabilities of exceeding the predefined limit states as a function of seismic ground motion intensity parameters. The obtained fragility curves are compared with the available fragility studies of the wide-beam, the flat slab and the conventional frame systems.

2. Modelling the Hysteresis Behavior of Wide-Beam Column Connections

The proposed study necessitated to define nonlinear springs that are representative of the behavior of the wide-beam column connections. For this purpose, some existing experimental research studies of the concentric wide-beam column connections are reviewed. Two interior wide beam connections by Benavent Climent et al. [1] and one exterior connection by Li and Kulkarni [6] are selected for calibration purposes. The simulations are performed in OpenSees [7]. The created models are composed of four rotational springs at the beam column joints, one at the end of the each beam and the column connecting to the joint. The existing geometric, material and loading characteristics of the experiment setups are assigned to the members in the models.

The Modified Ibarra-Medina-Krawinkler Deterioration Model with Peak-Oriented Hysteretic Response Material Model [7, 8] is selected as the nonlinear spring model. This model is an already defined material function (ModIMKPeakOriented) in Opensees. The model parameters are defined based on the simulation of the selected experiments. The performed extensive calibration study is a modified version of the calibration procedure of Haselton et al. [9].

The backbone of the Ibarra-Medina-Krawinkler Deterioration model is presented in Fig. 3. The backbone have four distinct zones. These are the elastic, hardening, post-capping and the residual zones. Every zone have the corresponding stiffness values. The interval from yielding point to capping point is defined as the plastic rotation. The model has four possible deterioration mechanisms which are basic strength deterioration, post-capping strength deterioration, unloading stress deterioration and the accelerated reloading stiffness deterioration. Further information about the selected hysteresis model could be found in the OpenSees element library and in Karaaslan [10].

The parameters of the Modified Ibarra-Medina-Krawinkler Deterioration Model with Peak-Oriented Hysteretic Response Material Model is first predicted using Haselton's calibration procedure, [9]. Although the selected prediction procedure provide estimates of the parameters, since the deficiencies of the wide-beam connections are not considered, corrections are needed in the parameters. For this purpose, a pushover analysis is held to accommodate a better calibration of the backbone curve. The monotonic behavior is calibrated through correction factors with the estimated values from the experimental results. The representative modification in the curves is presented in Fig. 4.

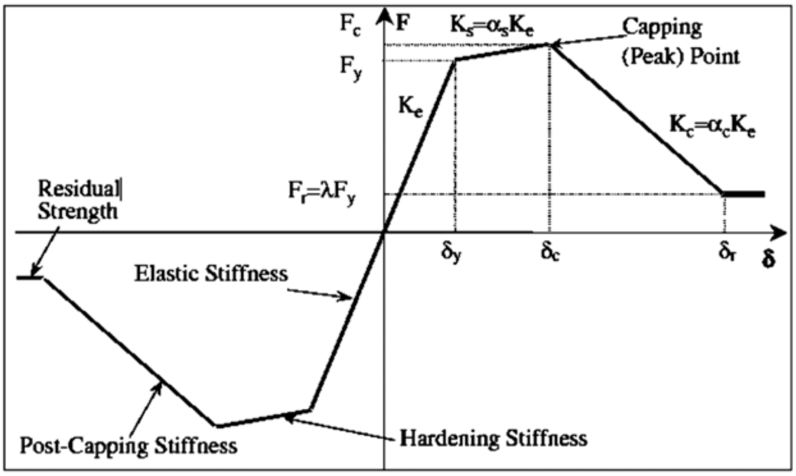

Fig. 3– Ibarra-Medina Krawinkler Deterioration Model Backbone Curve [7]

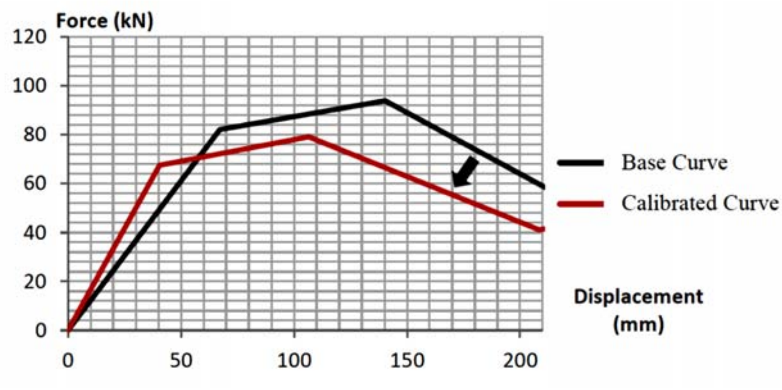

Fig. 4– Modification of the predicted backbone curve ($0.85M_y$, $1.4K_e$, $0.9\Theta_p$)

The last step in the calibration process is to determine the cyclic deterioration parameters. It is decided that the calibration of the post-capping strength deterioration parameter λ_C should have a value of 1.0-5.0 (medium deterioration), which gives an acceptable fit. Other deterioration parameters are kept as 1.0, which means full deterioration in each mechanism. The final form of the simulated hysteresis response of the specimen IL [1] is presented in Fig. 5. Further details about the calibration procedure and its results are available in Karaaslan [10].

3. Wide-Beam Building Frame Model

In order to investigate the seismic fragility of WBIJB block RC frame buildings, a generic planar frame model, which has been extracted from a 5-story, 4-bay (in both orthogonal directions) wide-beam building with regular and symmetric plan, is selected (Fig.6). The frame model conforms to the typical geometrical and material properties of Turkish wide-beam frame construction practice [5]. Accordingly, it is assumed that the joists span in wide-beam direction in the interior bays whereas they span in the opposite direction in the exterior bays. It is also considered that masonry infill walls are only present in the exterior bays except the ground floor. The details of the frame model are shown in Fig.7 together with all the applied loads.

The model building frame is composed of linear beam column elements, for which the lateral stiffness is defined in terms of gross sectional dimensions. Wide-beam joint behavior is simulated by the help of nonlinear rotational springs, which are characterized by the Modified Ibarra-Medina-Krawinkler Deterioration hysteresis model. The parameters of this model were previously calibrated in accordance with the available experimental data, as explained in the previous section. These spring elements are modeled with zero length elements constrained to the joint nodes in translational degree of freedoms.

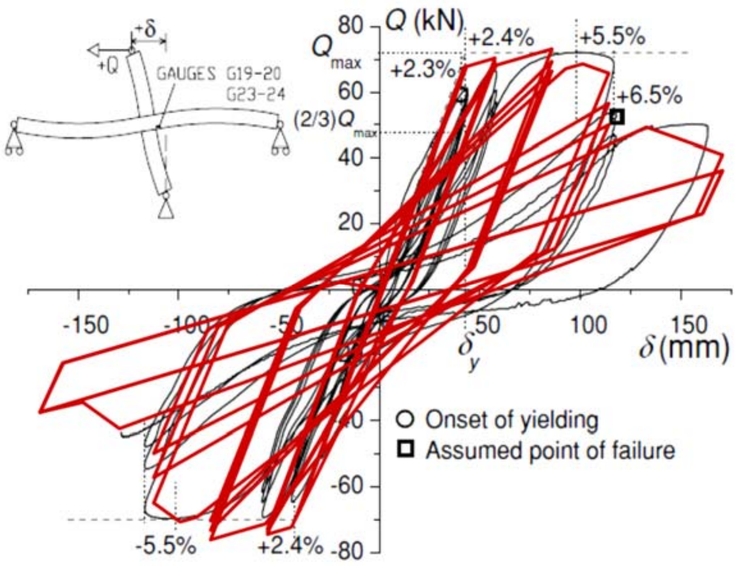

Fig. 5– Measured and simulated hysteresis responses of the specimen IL, [1]

The wide-beams and the columns have 32x50-cm and 60×30-cm cross sectional dimensions. In the frame model, concrete compressive strength and the yield strength of steel reinforcement are assumed as 30 MPa and 420 MPa, respectively. In addition the elasticity moduli of concrete and steel are considered as 31.8 GPa and 200 GPa, respectively. In order to take into account the structural contribution of the infill walls, the equivalent diagonal strut model, which was introduced by Mainstone [11] and then also used by the well-recognized international documents like FEMA 306 [12], is employed. The infill material properties, wall thickness, surrounding column and beam dimensions are the main parameters of the employed infill macro-model. The cyclic behavior of the infill wall is simulated through a piece-wise linear hysteretic model, for which the model parameters are calibrated through the experimental data by Dolsek and Fajfar [13].

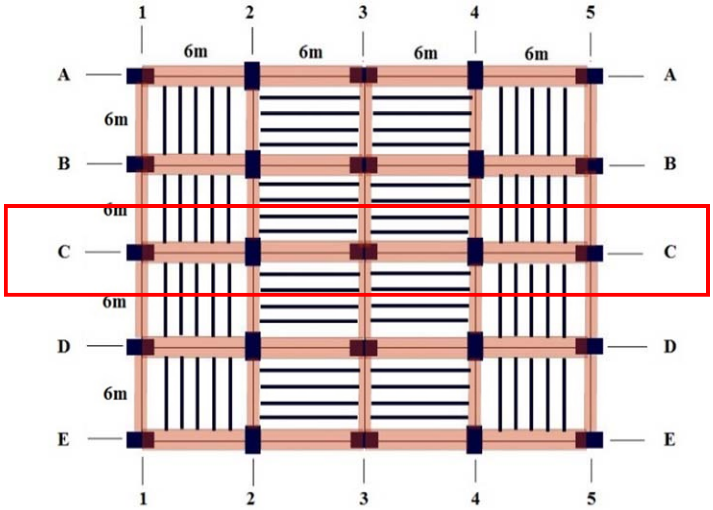

Fig. 6 – Floor plan of the wide-beam building and the selected frame

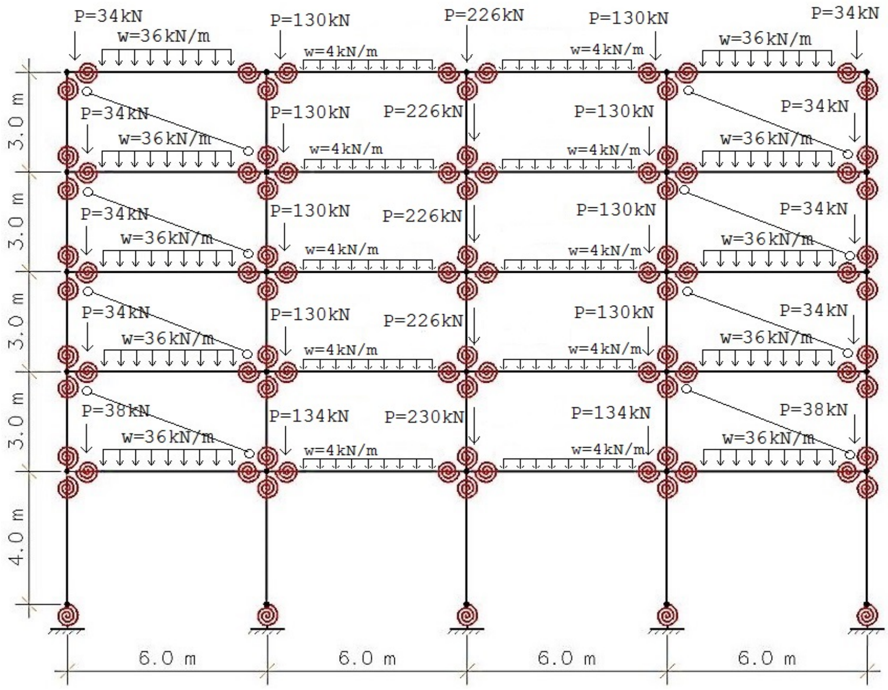

Fig. 7 – Selected planar wide-beam frame model

Seismic design of the building frame model is carried out in accordance with the Turkish Earthquake Code [14]. The building is assumed to be in the most severe seismic zone with the most unfavorable site conditions. After proportioning the members of the frame, drift check is carried out to ensure conformity to the code requirement

$$(\delta_i)_{max}/h_i \leq 0.02 \tag{1}$$

where $(\delta_i)_{max}$ is the maximum story drift and h_i is the story height of the typical i^{th} story. The results indicate that the considered frame model conform to the specified drift limit of the seismic code.

4. Fragility Analysis of the Frame Model

A fragility curve represents the relationship between the ground motion intensity and the probability of meeting the requirements of a certain performance limit. This probabilistic response is assumed to follow a lognormal distribution including the mean values of seismic demand and capacity together with their uncertainties, which can be shown as

$$P(LS/IM) = 1 - \left(\frac{\ln \eta_C - \ln \eta_{D/IM}}{\sqrt{\beta_{D/IM}^2 + \beta_C^2 + \beta_M^2}} \right) \tag{2}$$

where η_C is the median drift capacity for a specific limit state, $\eta_{D/IM}$ is the median drift demand given the ground motion intensity measure (IM), $\beta_{D/IM}$ is the standard deviation of the natural logarithm of drift demand for the same IM, β_C is the standard deviation of the natural logarithm of drift capacity and β_M is the uncertainty associated with the analytical modeling. In this study, median drift capacity is obtained for three different limit states by using pushover analysis results of the considered analytical model. These limit states can be regarded as LS1 (Immediate Occupancy), LS2 (Life Safety) and LS3 (Collapse Prevention). Median drift demand is obtained by conducting nonlinear time history analyses and fitting a power law equation for the obtained response statistics as a function of the considered ground motion intensity measure. Then the fragility curves are obtained by comparing the demand and the capacity for each limit state and calculating the probability of exceeding each limit state criterion as a function of the selected IM. The details of the fragility curve generation methodology are provided in the following sub-sections.

4.1 Quantification of Drift Capacity: Pushover Analysis

In fragility analysis, drift capacity is quantified in terms of the mean values of pre-defined performance limits (or limit states). The uncertainty involved in this quantification is also important and should be considered as seen in Eq. (2). There are two approaches to quantify the limit state values: first one is to use the readily available values proposed by well-recognized guidelines and standards like FEMA 356 (2000) or ASCE/SEI 41 (2013) [15, 16] and the second one is to obtain these values through pushover analysis of the analytical model. In the case of special types of building construction, like the wide-beam frame model in this study, it is more appropriate to use the second approach. Accordingly, pushover analysis is carried out for the wide-beam frame model by using a displacement controlled lateral static analysis procedure in OpenSees. The pushover curve obtained from the pushover analysis is given in Fig.8. The damage states of the rotational springs at the corresponding inter-story drift ratios are plotted on the pushover curve. The yielded springs (the spring that has reached its yield rotation capacity) are colored in green and the capping springs (the spring that has reached its capping rotation) are colored in yellow. Through the pushover analysis, no failing springs (which are colored in red) are encountered.

The limit states are quantitatively determined by considering the overall pushover behavior of the wide-beam frame building as shown in Fig.8. The immediate occupancy limit state (i.e. LS1) is defined as the first yield hinge mechanism. This limit state requires immediate occupancy after the earthquake with little or no plastic deformation. From the pushover curve it is observed that the rotational hinges at the connections yield as early as the 0.45% drift ratio. Therefore, LS1 is selected as 0.45% in this study. For the life safety limit state (i.e. LS2), the maximum lateral load capacity is taken as reference. The overall capping of the structure occurs at 1.15% drift ratio. Hence LS2 is assumed as 1.15% in this study. The collapse prevention limit state (i.e. LS3) is identified as the performance level where a capping hinge mechanism occurs in the system. At the drift ratio of 2.45%, all the ground story hinges reach their capping points and the structure exhibits immediate strength deterioration afterwards. Thus, LS3 is chosen as 2.45% in this study.

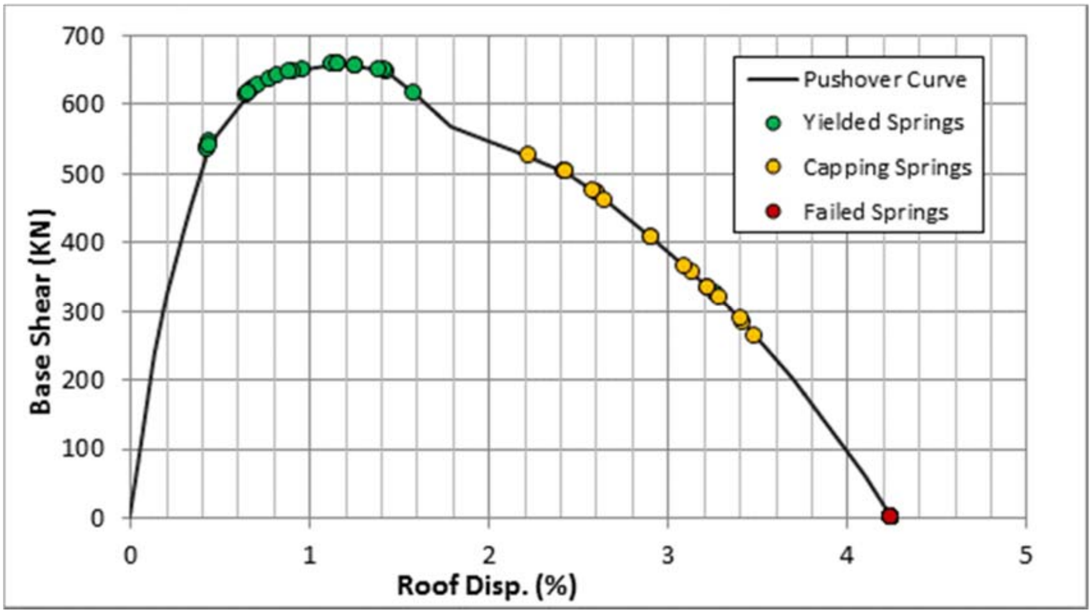

Fig. 8 – Pushover curve of the wide-beam building model and the damage states of the springs

4.2 Quantification of Drift Demand: Nonlinear Time-History Analyses

The wide-beam frame model is analyzed under a set of 100 earthquake excitations and story drifts are recorded by using the OpenSees platform. The earthquake ground motion set is composed of recorded data with varying strong ground motion parameters from different earthquakes and stations throughout the world to cover the whole range of seismic behavior for the case study analytical model. The full list of the ground motion set could be found in Karaaslan [10]. After time history analyses, the obtained response statistics is employed to develop demand prediction equations as a function of the selected ground motion intensity measures (IM). Fig.9 shows the plot of maximum drift versus different IMs and the corresponding best fit equations with their R^2 values. It is observed that peak ground velocity (PGV) gives the best fit with the highest R^2 value whereas peak ground acceleration (PGA) gives the worst fit. Hence the fragility curves are developed in terms of PGV in this study.

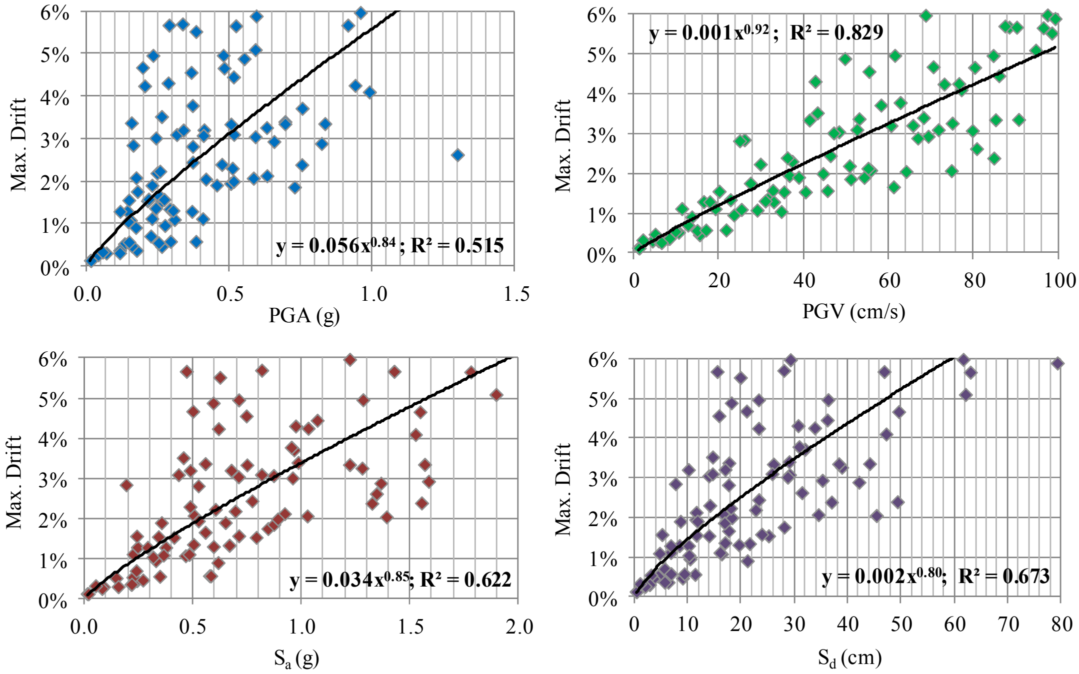

Fig. 9 – Plots of maximum drift versus various ground motion IMs and their best fit curves

4.3 Development of Fragility Curves

After quantifying the seismic capacity and demand for the case study building model, the last step is to generate the fragility curves for different limit states by using Eq.(2). As mentioned before, median drift capacities for LS1, LS2 and LS3 have been obtained as 0.45%, 1.15% and 2.45%, respectively. Median drift demands are calculated in terms of PGV by using the demand prediction equation in Fig.9. Standard deviation of the natural logarithm for drift demand ($\beta_{D/Sa}$) is calculated from the following equations.

$$\beta_{D/S_a} = \sqrt{\ln\left(1+s^2\right)} \tag{3}$$

$$s^2 = \frac{\sum\left[\ln Y_i - \ln Y_p\right]}{n-2} \tag{4}$$

In Eq.(3), s^2 is the square of the standard error. In Eq.(4), Y_i and Y_p can be defined as the observed drift demand and the predicted drift demand by the best fit equation, respectively. The parameters β_C and β_M are assumed as 0.3 in this study. The fragility curves obtained for the case study wide-beam model are presented in Fig.10.

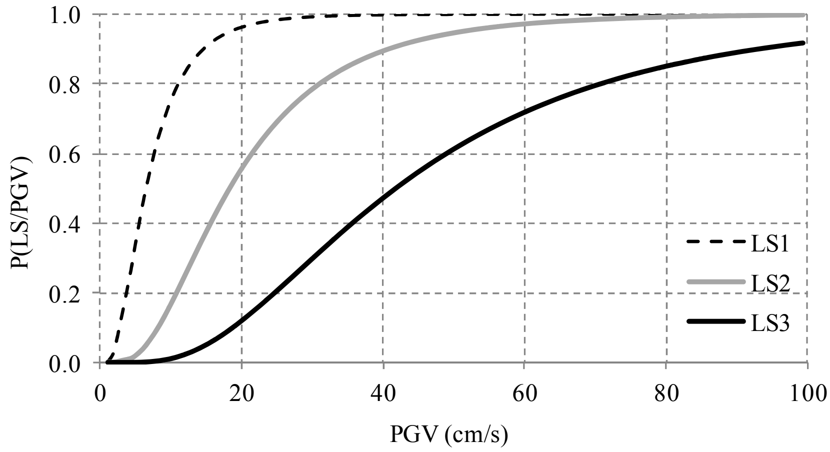

Fig. 10 – Fragility curves for the wide-beam model in terms of PGV

The case study building model with selected geometry and material parameters is a typical wide beam structure widespread in Turkey. Hence its fragility properties would be an indicator for its seismic performance when compared with other fragility studies on similar size and type buildings in high seismicity regions. Accordingly, the seismic fragility of the selected wide beam building model is matched with fragility curve sets of other types of buildings in the literature.

Since research on seismic vulnerability of wide-beam structures is very limited, there are not many alternative studies for comparison in the literature. The most suitable previous study belongs to Lopez Almansa et al. [17]), who studied the seismic vulnerability of RC buildings with one-way wide-beam slabs located in moderate seismicity regions of Spain. They selected two 3-story and four 6-story buildings to represent the vast majority of the existing buildings. The contribution of the infill walls was accounted for; accordingly, for each building, three wall densities were considered: no walls, low density and high density. The authors did not express the seismic vulnerability in terms of fragility curves, but they conducted dynamic analyses and determined the damage levels of their models in accordance with the definitions proposed by the project RISK-UE [18]. The 6-story 4-bay wide beam model with regular plan and low wall density seems to experience heavy damage for all of the five specific ground motion records they considered in their study. If the PGV values of these records are used to find the heavy damage state probability (i.e. beyond LS3) of the case study wide beam model, it is observed that the probability of experiencing heavy damage ranges between 75% - 99%. This means that heavy damage is quite possible to occur for all selected earthquakes according to both the vulnerability study carried by Lopez-Almansa et. al. and the fragility curves obtained in this study.

The second comparison is conducted between the fragility curve sets of the wide beam frame model in this study and the mid-rise RC moment resisting frame model developed by Ay and Erberik [19]. The number of stories, geometrical dimension and material properties of both models can be regarded as comparable. The RC frame model was also designed by considering the Turkish construction practice. Hence the comparison is intended to reveal the relative seismic performance of these two sub-classes of RC frame buildings with similar properties. The fragility curve sets are presented in Fig.11. The dashed lines represent the fragility curves obtained in Ay and Erberik whereas the solid curves are the ones obtained in this study. It is observed that although LS1 curves are close to each other, the wide-beam building model is more vulnerable at the other performance levels.

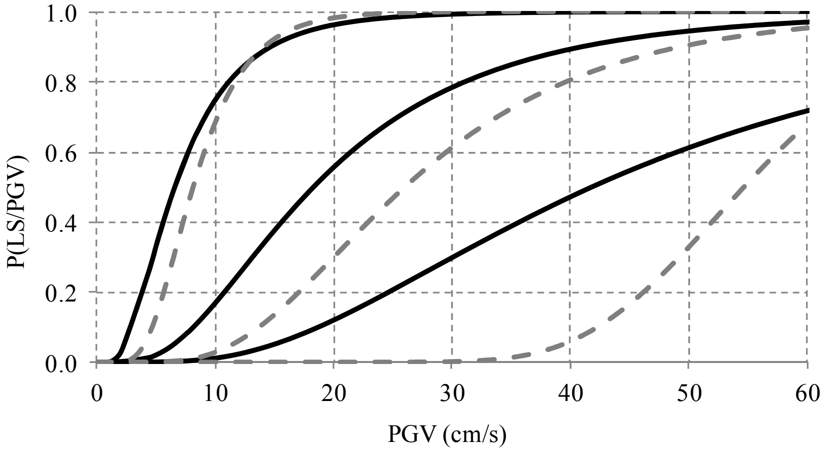

Fig. 11 – Comparison of fragility curves from this study and from Ay and Erberik (2008)

5. Conclusions

This research study puts forward the following conclusions:

- Research on the vulnerability of wide beam structures is very limited. However, there is a considerable number of existing wide beam structural frames and also, there is an increase in the construction of wide beam structural frames. In the current practice, such frames are entitled as reinforced concrete (RC) buildings.
- This study emphasize the importance of the simulations that consider the inherent characteristics of specific construction types such as wide beam frame structures. It could be concluded that to make the realistic estimations of the seismic vulnerabilities in specific construction types such as an approach should be used. The comparison of fragility curves show that the wide-beam frame structures should not be accepted to have the same response character with the conventional RC moment frames. Therefore, future regional loss estimation studies should use the specialized fragility curves for the wide-beam RC frames.
- A calibration study with the experimental research studies in the literature proved to be necessary to simulate the hysteresis behavior of wide-beam column connections realistically.
- Wide beam infill block frame structures in Turkey seem to be seriously vulnerable to seismic action although they conform to the current earthquake design requirements specified in Turkish earthquake code. The possible reason to this effects is the low lateral stiffness of such frames, which cause large inter-story drifts and therefore significant damage at the beam column connections. Measures to limit the period of these structures (or increasing the stiffness) could be a remedy to their systemic weakness.

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