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# NONLINEAR DYNAMIC ANALYSIS OF REINFORCED CONCRETE LARGE-PANEL BUILDING

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**Abstract:** A nonlinear dynamic analysis of large-panel reinforced concrete building (LPB) using LIRA-SAPR software is presented. LPBs, which became prevalent in the mid-20th century, have generally demonstrated good seismic performance during past earthquakes. However, the connection regions and specific structural characteristics still present challenges in accurately modeling and predicting their behavior under seismic loading. Conventional linear models often inadequately represent the complex nonlinear behavior such as strength and stiffness degradation and connection failures inherent in LPBs, highlighting the need for more sophisticated analytical approaches. A detailed nonlinear analysis methodology was applied to a full-scale reinforced concrete precast LPB, investigating various parameters including material properties, damping variation, and their effects on interstory drift ratios and displacement patterns. The analysis revealed significant differences in seismic response, particularly for lower concrete grades, emphasizing the critical role of accurate material characterization. Displacement and acceleration distributions exhibited directional dependencies, aligning with results from existing literature and full-scale testing. Despite the strengths of the LIRA-SAPR software, certain limitations were identified, such as the inability to modify the standard hysteresis model, which lowers the accuracy of the simulation of strength and stiffness degradation. The findings suggest the necessity of modifying existing building codes to include specific acceptance criteria and updated analysis procedures tailored to the unique behaviors of LPBs. Such modifications are essential for developing effective tools for seismic performance evaluation and enhancing the resilience of these structures.

**Keywords:** Large-Panel Buildings, Nonlinear Dynamic Analysis, Seismic Performance Evaluation

# НЕЛИНЕЙНЫЙ ДИНАМИЧЕСКИЙ АНАЛИЗ КРУПНОПАНЕЛЬНОГО ЖЕЛЕЗОБЕТОННОГО ЗДАНИЯ

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**Аннотация:** Представлен нелинейный динамический анализ крупнопанельного железобетонного здания (КПЗ) с использованием программного комплекса ЛИРА-САПР. Крупнопанельные здания, получившие широкое распространение в середине прошлого века, в целом продемонстрировали хорошую сейсмостойкость во время прошедших землетрясений. Однако соединения панелей и специфические конструктивные особенности по-прежнему создают проблемы для точного моделирования и прогнозирования их поведения при сейсмических воздействиях. Обычные линейные модели часто неадекватно отражают сложное нелинейное поведение, такое как деградация свойств и разрушение соединений, присущее КПЗ, что подчеркивает необходимость применения более сложных аналитических подходов. Детальная методология нелинейного анализа была применена к железобетонному КПЗ с учетом различных факторов, включая свойства материалов, вариацию демпфирования и их влияние на значения междуэтажных перекосов и характер деформации. Анализ выявил значительные различия в сейсмической реакции, особенно для более низких марок бетона, что подчеркивает критическую роль точного определения характеристик материала. Распределения перемещений и ускорений демонстрируют зависимость от направления воздействия, что согласуется с результатами, представленными в литературе и натурными испытаниями. Несмотря на сильные стороны программного

обеспечения ЛИРА-САПР, были выявлены некоторые ограничения, такие как невозможность модификации модели гистерезиса, что ограничивает точное моделирование деградации прочности и жесткости, характерной для данного типа сооружений. Полученные результаты свидетельствуют о необходимости модификации существующих строительных норм и правил для включения в них специальных критериев оценки сейсмостойкости и улучшенных аналитических процедур, учитывающих уникальные особенности поведения КПЗ. Такие изменения важны для разработки эффективных инструментов прогнозирования сейсмической повреждаемости данных сооружений.

**Ключевые слова:** крупнопанельные здания, нелинейный динамический анализ, оценка сейсмостойкости

## INTRODUCTION

Large Panel Buildings (LPBs) have been a significant advancement in construction technology since their inception in the mid-20th century [1,2]. Originally developed to meet the urgent post-war housing demands, LPBs revolutionized the construction industry with their modular approach, enabling rapid assembly and cost-effective production [3,4]. The precast concrete panels used in LPBs offered numerous advantages, including enhanced durability, improved thermal performance, and consistent quality control. Over the decades, LPBs have evolved to incorporate advancements in materials science and construction techniques, solidifying their place in both residential and commercial architecture.

Despite their widespread use, LPBs have faced scrutiny regarding their seismic performance, particularly in regions prone to earthquakes. The connection regions and large precast panels characteristic of LPBs pose unique challenges under earthquake loading [5,6]. Historical seismic events have demonstrated varying degrees of structural vulnerability, highlighting the need for a deeper understanding and enhancement of LPB seismic resilience [7,8]. Studies have shown that the performance of LPBs during earthquakes is influenced by factors such as panel connections, overall structural configuration, and the quality of construction materials [9,10].

The analytic study of LPB seismic performance has become a crucial area of research, aiming to predict and mitigate potential failures. Advanced analytical methods, including nonlinear dynamic analysis, have been employed to simulate the behavior of LPBs under seismic loads [11,12]. These studies provide insights into the stress distribution, deformation patterns, and potential failure modes of LPB structures.

To accurately assess and improve the seismic performance of LPBs, the development of relevant and reliable analytical models is essential. These models must account for the complex interactions between panels, connections, and the overall structural system. They should incorporate nonlinear material properties, dynamic loading conditions, and potential degradation. The use of sophisticated software tools, such as LIRA-SAPR [13], enables the creation of detailed and robust models that can simulate real-world seismic scenarios. These models serve as a foundation for developing effective retrofitting strategies and informing building codes and standards.

This study aims to conduct a comprehensive nonlinear dynamic analysis of a reinforced concrete precast large-panel building using the LIRA-SAPR software. By simulating the seismic response of the LPB structure, this research seeks to identify critical stress points, deformation behaviors, and potential failure mechanisms. The findings will contribute to the ongoing efforts to develop robust tools for seismic performance evaluation, such as fragility curves, and calls for modifications to existing building codes [14].

## METHODS

The object of the study is a typical residential apartment building of large-panel type series 1-092, located in Vladikavkaz city (Figure 1): 9-

story 1-section building. The story height is 3 meters. External nonbearing walls are 300 mm thick panels, internal bearing walls are 160 mm thick single-layer panels, partitions are 80 mm thick. Floor panels 160 mm thick. Concrete grade – B15 ($f'_c$ = 8.5 MPa, $E_c$ = 24000 MPa), reinforcement – A-I ($R_s$ = 240 MPa, $E_s$ = 200000 MPa).

A visual inspection of the building revealed several characteristic defects, including partial absence of joint filling, corrosion of joint metal, and poor or complete lack of joint filling (as shown in Figure 3). To account for these defects within the limitations of the software, a concrete grade of B7.5 ($f'_c$ = 4.5 MPa, $E_c$ = 16000 MPa) was also employed in the analyses for the joint regions.

The horizontal and vertical panel joints are shown in Figure 2.

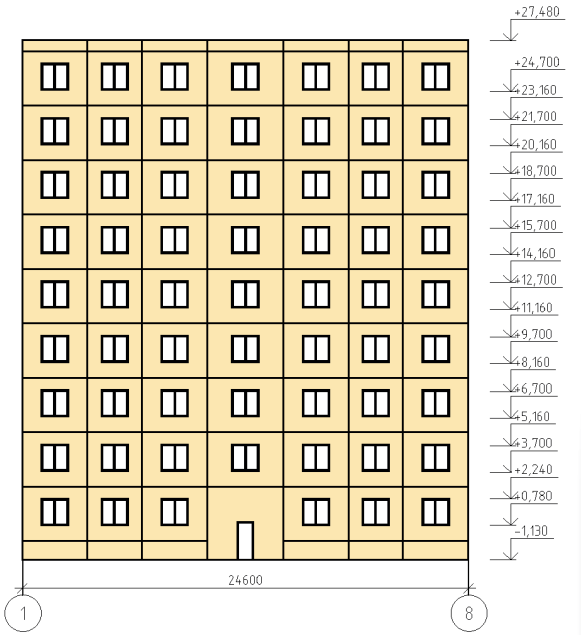



*Figure 1. Object of study*

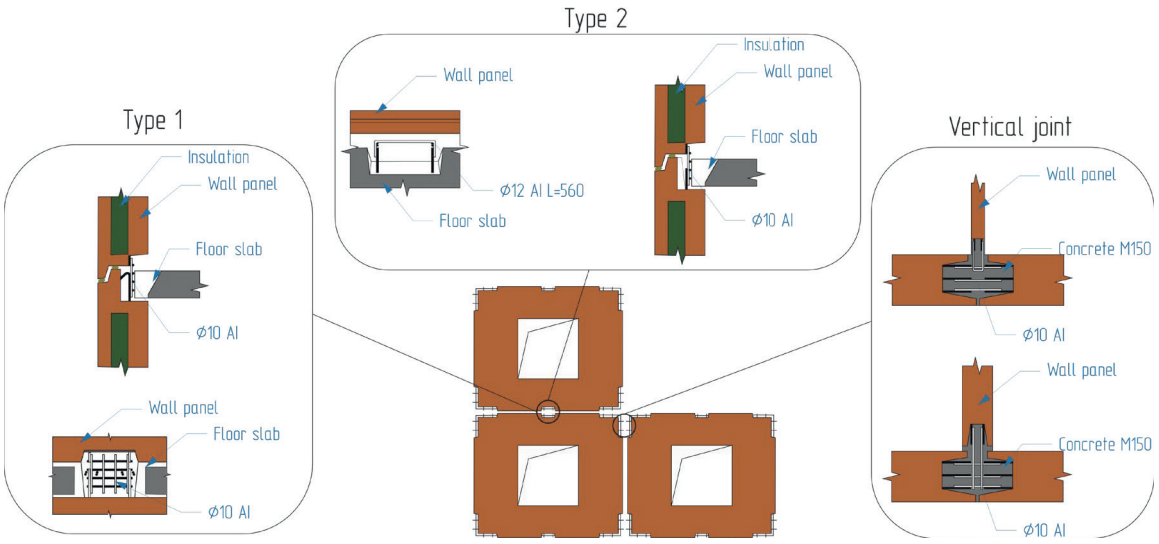



*Figure 2. Details of the building*



*Figure 3. Defects of the building*

**Modeling assumptions**

In the modeling of large-panel precast structures, it is conventional to assume that the solid panels exhibit linear elastic behavior. This assumption is justified by the anticipated function of these elements within the structure's overall seismic response. Typically, wall panels are considered to behave in an elastic-brittle manner with respect to in-plane normal forces. Any potential nonlinear or inelastic behavior is often deemed negligible in the panels themselves and is concentrated in the connection regions, where it can be more effectively modeled and analyzed [11].

The following assumptions are made for the joint element [15]:

– all points of the horizontal section remain in one plane after the force is applied (plane sections remain plane);

– for the tensile zone of the section the tensile strength of concrete is not taken into account;

– normal compressive stresses distribution along the section is linear or bilinear.

**LIRA-SAPR implementation**

The finite element method software LIRA-SAPR is used for numerical analysis.

Nonlinear behavior of the connection region is modeled by a combination of two finite elements (FE) (Figure 4) [16]:

– Shell Element (FE 259) representing stress distribution in the connection region,

– 2-mode link element (FE 255) for modeling shear keys.

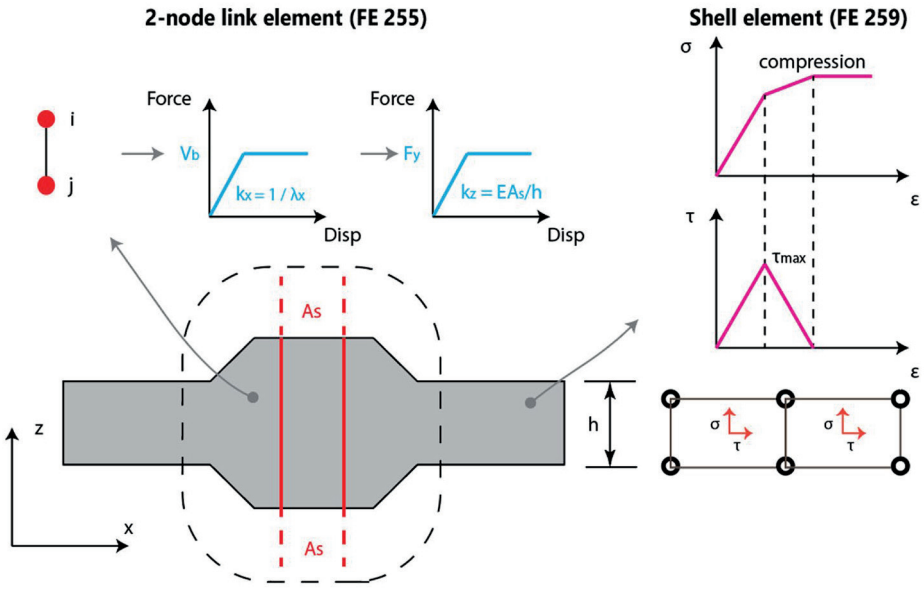



*Figure 4. Finite element implementation of the connection region*

The precast walls and floor panels are modeled with elastic shell element FE 44.

*2-mode link element (FE 255) parameters*

According to section B4 of SP 335.1325800.2017 [17,18], the design shear strength $V_{kb}$ of one shear key of the is taken equal to the smallest of the force values $V_{sh,b}$; $V_{c,b}$; $V_{crc,b}$, corresponding to the destruction of the shear key, respectively, from shear, crushing and shear cracks opening:

$$V_{sh,b} = 1.5 R_{bt} A_{sh} \quad (1)$$

$$V_c = R_{b,loc} A_c \quad (2)$$

$$V_{crc,b} = 0.7 R_{bt} A_j \quad (3)$$

where $R_{bt}$ is design tensile strength of grouting concrete; $R_{b,loc}$ is crushing strength, taken equal $1.5R_b$ – for single keys, $0.8R_b$ – for multikey connections; $A_{sh}$ is shear area; $A_c$ is crushing area; $A_j$ – cross-sectional area of the connection tributary to each shear key:

$$A_j = s_{bt} b_{mon} \tag{4}$$

where $s_{bt}$ is distance between shear keys; $b_{mon}$ is wall thickness of the grouting area.

Stiffness along *Z*-axis is represented by bilinear elastoplastic force-displacement relationship, elastic stiffness is determined by:

$$k_z = EA_s / h \tag{5}$$

where $E$ is elastic modulus of the rebar; $A_s$ is rebar cross-sectional area; $h$ is rebar length (equal to the joint height).

Stiffness along *X*-axis is represented by bilinear elastoplastic force-displacement relationship, elastic pliability is determined by:

$$\lambda_x^1 = \frac{l_{loc}\left(\frac{1}{E_{b,\omega}} + \frac{1}{E_{b,mon}}\right)}{A_{loc}} \tag{7}$$

where $A_{loc}$ is loaded area; $l_{loc}$ is shear key height (recommended $l_{loc}$ = 250 mm); $E_b$ is elastic modulus of concrete precast element; $E_{b,mon}$ is elastic modulus of grouting concrete.

The pliability after shear cracks occurring (exceeding the shear force $V_{crc,b}$):

$$\lambda_x^2 = \frac{6}{d_s n_s}\left(\frac{1}{E_{b,\omega}} + \frac{1}{E_{b,mon}}\right) \tag{8}$$

where $d_s$ is connection rebars diameter; $n_s$ is number of the connection rebars.

*Shell Element (FE 259) parameters* [19,20]

During short-term compression for a mortar with a compressive strength of 1 MPa or more, with a joint thickness of 10–20 mm, pliability coefficient of the mortar bed joint $\lambda_m$, $mm^3/N$:

$$\text{for } \sigma_m \le 1.15R_m^{2/3} \quad \lambda_m = 1.5\cdot 10^{-3} R_m^{-2/3} t_m \tag{9}$$

$$\text{for } 2R_m^{2/3} \ge 1.15R_m^{2/3} \quad \lambda_m = 5\cdot 10^{-3} R_m^{-2/3} t_m \tag{10}$$

Pliability coefficient of the joint:

$$\lambda_{c,pl} = (\lambda_m + \lambda_m^{'} + \frac{h_{sl}}{E_b})\frac{B_{pl}}{B_{pl} - h_v} \tag{11}$$

where $\lambda_m^{'}$ and $\lambda_m$ are upper and lower mortar bed joint pliability coefficients; $h_{sl}$ is floor panel height; $B_{pl}$ is contact areas width; $h_v$ is joint width between floor panels.

Strain:

$$\varepsilon = \frac{\lambda_i \sigma_i}{h_v} \tag{12}$$

Stresses:

$$\sigma_m^1 = 1.15R_m^{2/3} \tag{13}$$

$$\sigma_m^2 = 2R_m^{2/3} \tag{14}$$

$$\sigma_m^y = 1.01\sigma_m^2 \tag{15}$$

To reduce the computational time the optimal finite element mesh size was determined by considering three mesh configurations: 0.1 m, 0.2 m, and 0.4 m. In the case of panels with openings, which complicate the computation by inducing stress concentrations – these panels were replaced with solid panels of reduced stiffness. The reduction factor was calculated as the ratio of the wall area with openings to the wall area without the openings. As a result, the finite element mesh of the wall panels was set to 0.4 × 0.4 m, and the finite element mesh of the connections was set to 0.4 × 0.1 m.

The resulting finite element model in LIRA-SAPR is presented in the Figure 5.

Self-weight and live load according to Russian codes were applied.

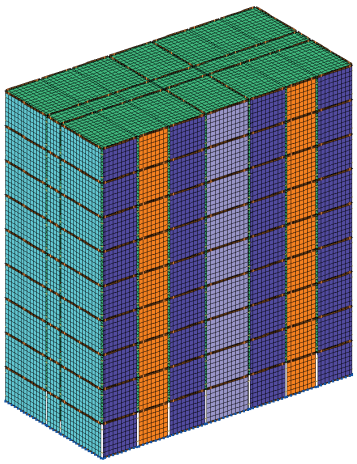


*Figure 5. Finite element model of the building*

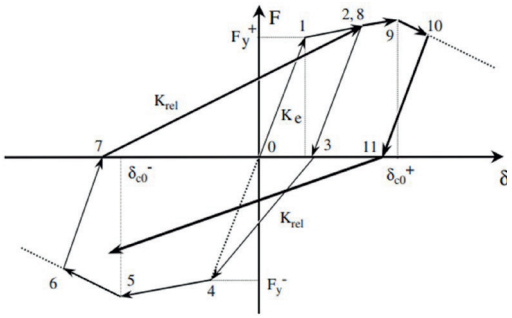



*Figure 6. Basic rules for peak-oriented hysteretic model*

The analysis was performed using an explicit integration of the equation of motion [21]:

$$M\ddot{x} + C\dot{x} + Kx = p(t) \qquad (21)$$

where $x = u$ is the unknown vector of nodal displacements; $\dot{x} = v$ is the nodal velocity vector; $\ddot{x} = a$ is the nodal acceleration vector; $M$ is the mass matrix; $C$ is the damping matrix; $K$ is the stiffness matrix; $p(t)$ is the vector of applied loads.

*Analysis parameters:*

– Newmark-beta method (average constant acceleration with $\gamma = 0.5$ and $\beta = 0{,}25$) is used for direct integration of the equations of motion.

– Modified Newton-Raphson method is used for solving nonlinear equations at each time step.

– The results from the natural dynamic testing of the full-scale LPB reveal considerable temporal variations in the structure's damping characteristics [15]. To examine the sensitivity of the structural response, damping ratios $\xi$ of 2%, 3%, 4%, and 5% were employed in the analyses.

– LIRA-SAPR's standard peak-oriented hysteretic model (Figure 6) is applied. The detail description of the employed model is presented in [22].

It is important to note that in the LIRA-SAPR software, the parameters of the standard hysteresis models cannot be modified or changed. While this limitation may be adequate for typical structures, it represents a significant shortcoming when analyzing precast panel structures, which exhibit specific strength and stiffness degradation behaviors. The inability to customize hysteresis rules restricts the accuracy of the simulation, particularly for structures with unique dynamic characteristics. Future releases of the software should address this issue by incorporating flexible hysteresis options, enabling more precise modeling of the complex behaviors inherent in precast panel systems.

– Total integration time is 20 sec.

– Integration step is 0.01 sec.

The model is subjected to two component of ground motion of the 1988 Spitak, Armenia earthquake (Figure 7–8). An earthquake registering a surface wave magnitude of 6.8 and a maximum MSK-64 intensity of X (Devastating) hit the northern part of the Armenian Republic of the Soviet Union on 7 December 1988, resulting in thousands of deaths and injuries [23].

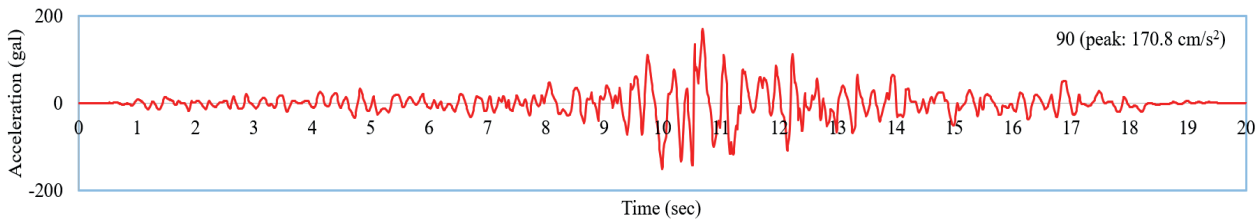



*Figure 7. Ground acceleration record (Spitak, Armenia, 1988, Gukasian, component 000)*

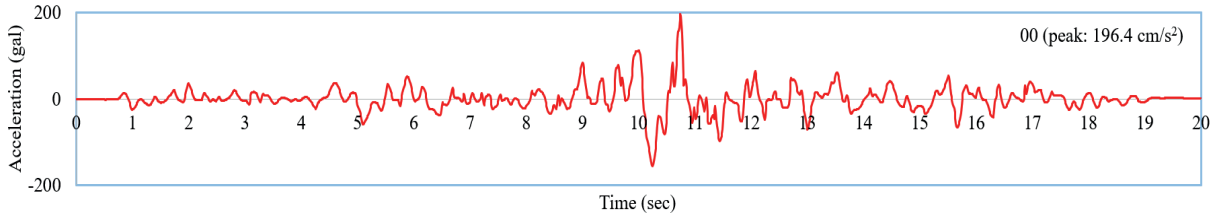



*Figure 8. Ground acceleration record (Spitak, Armenia, 1988, Gukasian, component 090)*

## RESULTS AND DISCUSSION

The natural period of the structure is 0.37 s in Y-direction.

The maximum displacements, shown in Figure 9, occur in the Y-direction for both materials: 24 mm for B7.5 and 20 mm for B15, compared to 7.31 and 4.59 mm in the X direction, respectively. Figure 10 shows the distribution of the maximum acceleration, with peaks of 3.1 m/s² in the X direction and 2.44 m/s² in the Y direction at the top story.

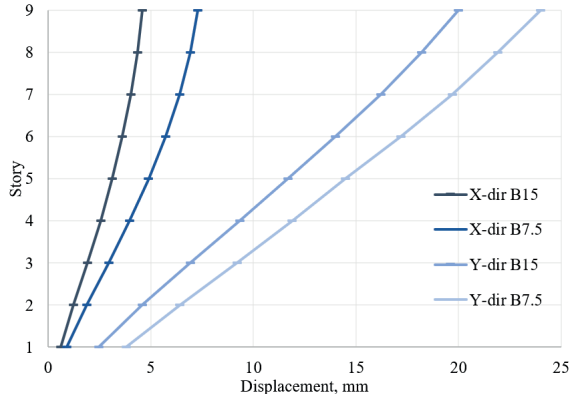



*Figure 9. Maximum displacements (ξ = 5%)*

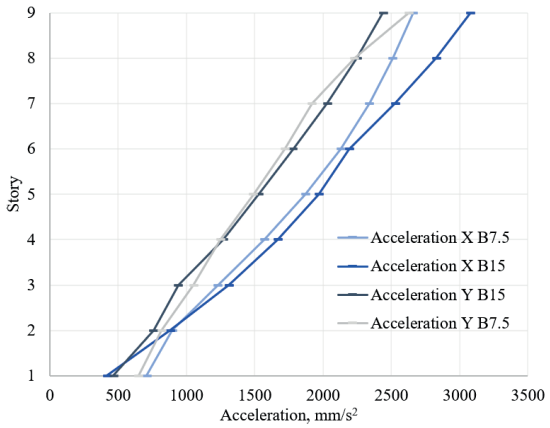



*Figure 10. Maximum accelerations (ξ = 5%)*

These results are consistent with findings reported in the literature and with the outcomes of natural testing conducted on full-scale buildings [15,24].

The corresponding graph of the interstory drift ratio (Figure 11) exhibits a similar pattern across different damping values, with the maximum drift occurring at the first story. However, the differences in these values are significant, with variations up to 25%. This notable discrepancy underscores the importance of accurately determining the appropriate damping ratio, which necessitates thorough investigation in future research.

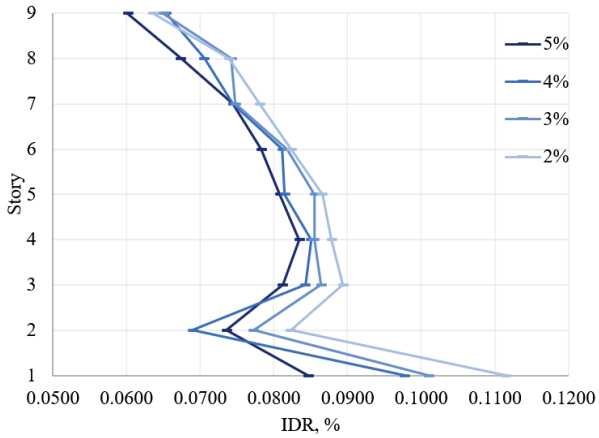



*Figure 11. IDR for different damping ratios*

The interstory drift ratio (IDR), which measures the relative horizontal displacement between adjacent floors of a building, is a critical indicator of structural damage [25]. The IDR for different concrete grades is illustrated in Figure 12. The analysis reveals that the seismic response for the lower concrete grade is significantly higher, with an IDR of 0.132 % compared to 0.085 % for the higher grade. This notable difference underscores the critical

importance of meticulously investigating the material parameters of buildings in future research. Despite the disparity in IDR values, the overall response in both cases remains relatively low when compared to the commonly accepted ranges for performance-based design as specified by building codes [26]. However, it is important to note that there are currently no established acceptance criteria specifically tailored for LPBs, highlighting the urgent need for the development of such criteria. These should account for the unique behavior and fragility characteristics of this structural type.

The findings presented in Figure 12 are consistent with post-earthquake investigations of LPBs following the Spitak earthquake, where none of these buildings were destroyed or suffered significant damage [27]. This correlation further emphasizes the necessity for a comprehensive framework to evaluate and enhance the seismic resilience of LPBs

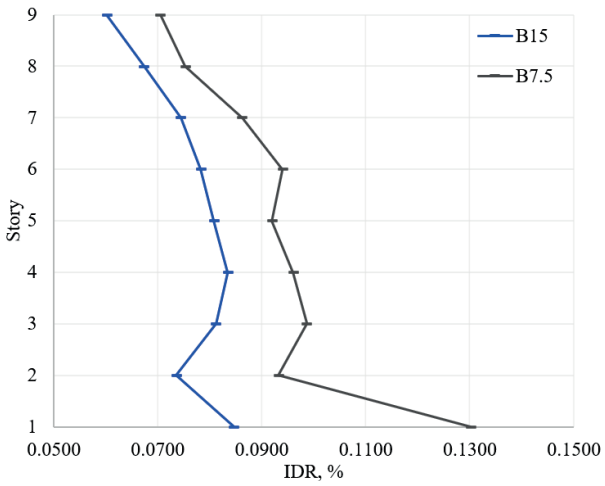



*Figure 12. IDR for B7.5 and B15 concrete grades*

One effective approach for such an evaluation is the development of fragility curves, which depict the probability of reaching various performance levels at different ground motion intensity levels [28]. These curves are crucial in quantifying the seismic vulnerability of structures, as they provide a statistical relationship between seismic demand and structural performance. Fragility curves are typically developed through a statistical analysis of the results obtained from Incremental Dynamic Analysis (IDA) [29].

In IDA, the structure is subjected to a suite of ground motions that are incrementally scaled in intensity, allowing for a detailed assessment of its nonlinear response and failure modes. An example of IDA for a selected ground motion scaled for 6 levels of intensity (from 0.1 g to 0.6 g) is presented in Figure 13. This figure illustrates how the structural response evolves with increasing ground motion intensity, highlighting critical thresholds where performance levels transition from minor damage to potential collapse.

It is important to note that as the intensity level increases, the dispersion in the response also becomes more pronounced. These factors contribute to longer computational time and may introduce inaccuracies if not carefully managed. Therefore, it is imperative that the analysis parameters of the IDA, including mesh size, time step, and convergence criteria, are meticulously calibrated beforehand to ensure reliable results.

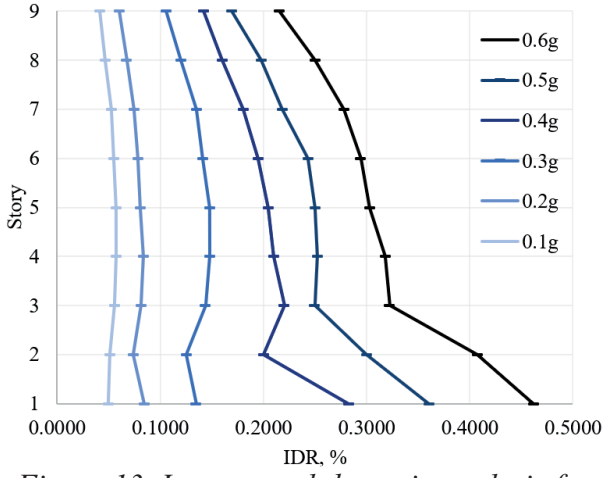



*Figure 13. Incremental dynamic analysis for selected ground motion (B15, ξ = 5%)*

To successfully implement these findings and develop robust tools for seismic performance evaluation, targeted modifications to existing building codes are necessary [14,30]. These modifications should include the establishment of specific acceptance criteria for large-panel

buildings (LPBs) that account for their unique structural behavior and fragility under seismic loading.

## CONCLUSION

This study has undertaken a detailed nonlinear dynamic analysis of large-panel buildings (LPBs) using the LIRA-SAPR software, focusing on accurately modeling the seismic response of precast concrete structures. The methodology developed in this research allowed for the simulation of complex behavior inherent in LPBs, particularly under seismic loading. The analysis revealed key insights into the seismic performance of these structures, with significant differences observed in the interstory drift ratios and displacement patterns depending on the material properties.

One of the critical findings was the considerable variation in seismic response based on different concrete grades of the connections, with lower-grade concrete exhibiting significantly higher interstory drift ratios. In addition, the notable discrepancy in response for different damping ratios (up to 25%) was observed for the model. These variations highlight the importance of precise material characterization in predicting structural behavior of LPBs. The results align well with previous studies and the outcomes of natural testing on full-scale buildings, providing a strong validation of the analysis conducted.

However, the study also identified limitations within the LIRA-SAPR software that could impact the accuracy of such analyses. Notably, the inability to modify or change standard hysteresis models posed challenges in accurately representing the strength and stiffness degradation specific to precast panel structures. Despite these constraints, the overall findings of the study offer valuable insights into the seismic behavior of LPBs, emphasizing the importance of detailed nonlinear analysis in understanding and predicting their performance.

In summary, the research successfully applied a nonlinear analysis methodology to LPBs using LIRA-SAPR, yielding critical findings on the seismic response of these structures. While some software limitations were identified, the results contribute to the understanding of LPB behavior under seismic loads and underscore the necessity of code modifications to support the development of effective seismic performance evaluation tools.

## ACKNOWLEDGEMENTS

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