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CONTRIBUTION TO THEORETICAL AND EXPERIMENTAL ANALYSIS OF LARGE PANEL STRUCTURES SUBJECTED TO EARTHQUAKE

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ABSTRACT

This paper reviews experimental and numerical seismic response of a new precast large panel system developed by JIMPROS – Yugoslavia. The investigations concerning the interaction of connection elements between bearing precast wall panels were performed on a seven floor building with non-symmetrical floor plan, founded on loess soil in Belgrade. The numerical model for JIMPROS large panel system subjected to earthquake loading was proposed considering experimental results. Space (3D) analysis for the global structure was performed using linear and non-linear structural analysis computer programs. Extensive experimental and non-linear numerical analysis was performed on reinforced concrete panels for investigating local effects in joint panel connections. For modeling precast reinforced concrete panels, 2D finite element with the corresponding hysteresis behaviour was proposed. The element can describe both flexural and shear response due to seismic loads. A hysteresis model can involve degrading stiffness and strength with softening and pinching. Conclusions derived from the theoretical and experimental analysis have practical effect on rational design and analysis of elements and connections in the large panel system for the precast buildings.

KEYWORDS

Large panel structures, precast wall panels, panel joints, in-situ building experiment, FEM analysis.

INTRODUCTION

Modern structural design of precast-reinforced multistory office and residential buildings in regions affected by strong earthquake motions is based on a space bearing system assembled of beam and panel elements. According to the choice of the vertical elements in the bearing system, we have the flexible frame-type structures and "rigid" frameless large panel systems. The comparison in performance due to earthquake excitation between these two systems, similar in layout, has shown the advantages of using large panel structures. It is well known fact that during last fifteen years, multistory large panel buildings subjected to earthquakes (in Russia, Yugoslavia and Romania) suffered much less damage compared to other building types (frame type, mixed, masonry and others). There are several reasons for its high earthquake resistance: very effective load distribution among a greater number of comparatively poorly loaded elements (precast facade elements); the space system rigidity which ensures better force redistribution among the bearing elements; non-elastic energy dissipation and non-linear behaviour due to the work of joints, lintels above openings and interaction between the foundation and the soil.

In areas requiring design for seismic loading, ductility becomes an extremely important consideration. However, there exists a widely spread opinion that large panel-systems are not distinguished by a high degree of ductility and they are inferior toward the frame type structures. To overcome this shortcoming, and ensure ductile behaviour, special attention was given to details such as the shape and size of panels and their connections. Procedures for evaluating the available ductility of the bearing elements and their connections are of significant importance to enable designers of the large panel systems to ensure that these structures have adequate available ductility to match the required one. This paper is a very brief presentation of recent experimental and theoretical research work in evaluating ductility of the large panel precast system that is in use in Yugoslavia.

EXPERIMENTAL INVESTIGATIONS

An integrated experimental and analytical research program has been performed at the Civil Engineering Faculty in Belgrade and the laboratory for strength testing, TSNIIEP Moscow, as part of the project: "Design and construction of a new precast large panel system", developed by JINPROS (Business Association of Yugoslav producers of precast components and industrialized construction). In order to investigate the capability of such structural system under seismic conditions, and to develop improved methods for structural design and detailing, coordinated series of experimental and analytical studies have been conducted on a JINPROS seven-story large panel building in Belgrade. The photography of such a residential building and its floor plan are shown in Fig. 1.

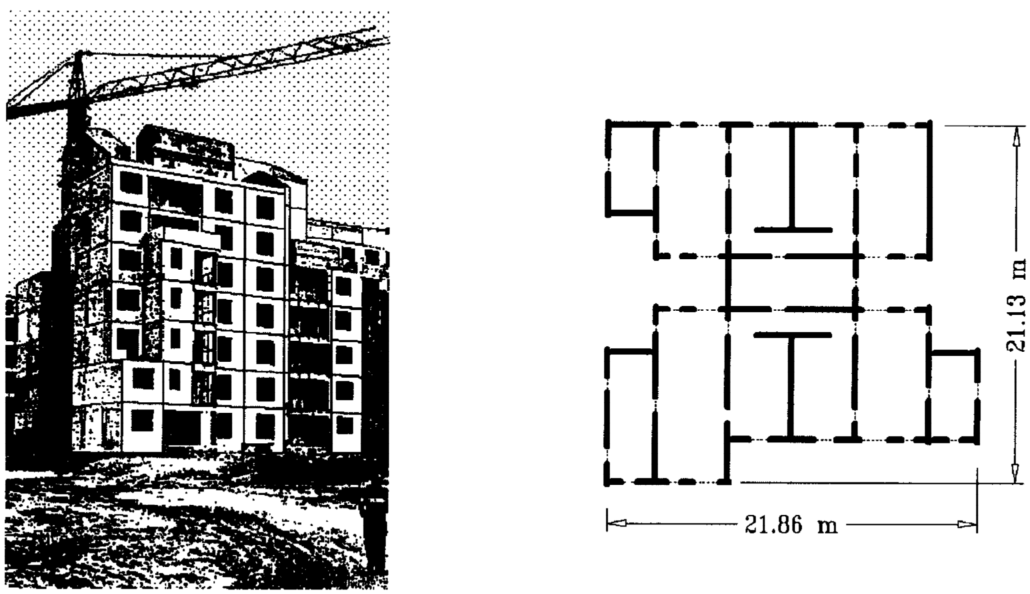

Fig. 1. Seven-story large panel building and its floor plan

The residential building has been designed for lateral loads according to Yugoslav earthquake codes, for a degree of seismic intensity equal to 8 (MSK-scale). The stresses and the corresponding reinforcement in the bearing panels have been computed by elastic-linear analysis under code seismic forces. The full scale experimental tests were carried out by TSNIIEP from Moscow, with a powerful vibrating equipment for inducing horizontal vibrations. The moment of unbalanced masses of 6 vibrators (23.40 kNm) was able to create the exciting force up to 1200 kN under the frequency over 3.5 Hz. The equipment placed on the top floor of the building induced vibrations in two orthogonal directions - lateral and longitudinal. Despite the large size and mass of the building (3500 kN) and high energy dissipation due to the soil-building interaction, very high magnitudes of vibrations were registered. It was confirmed both by the decrease in resonance frequency for approximately 30% and the appearance of the damages in the structural elements.

The structure of the building, with non-symmetrical floor plan (Fig. 1) and storey height of 2.8 m, has been assumed as a frameless structure in which the bearing internal walls were designed to withstand the vertical and horizontal loads. These walls were assembled with comparatively short precast panels of thickness 20 cm. The required reinforcement was obtained due to the combined effect of axial-flexural and shear forces acting at the critical regions of the panels. The adjacent precast internal panels in the plane of loading and perpendicular to the plane of loading, were connected along the vertical sides locally with two steel joint plates welded on site. These joint plates were designed to resist only lateral loads. The dynamic experimental tests, performed on shaking table in Skopje (Velkov and Bozinovski 1984), of the panel fragment containing joint plate, proved their high compliance (Fig. 2). The bearing capacity was achieved under load reversal displacement exceeding 7 mm (Fig. 2). The horizontal joints between the floor slabs and the wall panels were designed as tie-beams cast on site, which also prevented the load reversal displacements of the wall panels in the vertical direction. The floor and roof slabs were designed as precast simply supported, one-way span flat plates of thickness 16 cm. In order to achieve the horizontal continuity and in-plane monolithic action, the adjacent precast plates were connected with special steel plates joined by welding. The external wall panels were not designed to withstand the lateral loads. The entire building bearing system was supported on the flat reinforced-concrete slab of constant thickness, founded on loess soil with low bearing capacity.

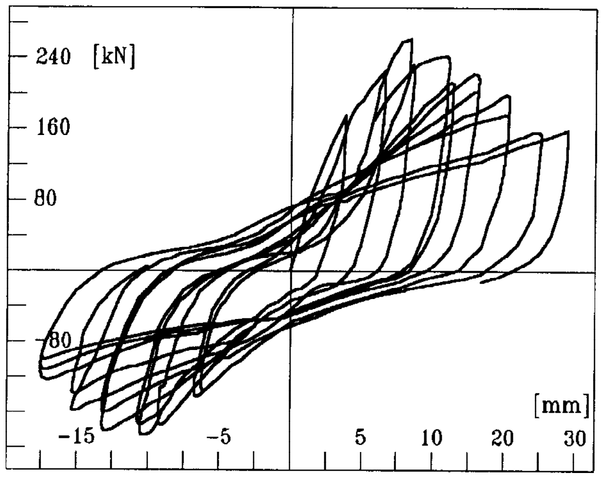

Fig. 2. Relationship force-shear for the joint panels

One of the main research efforts was to examine the performance of these joint connections between the precast concrete panels. It is well known that both the structural stiffness of the entire system and the distribution of the stresses within the members considerably depend on the compliance of the vertical joints. The experimental investigations have shown that large panel frameless systems are sensitive to the change of joint compliance only within the range ($\lambda_I = 10^{-6}–10^{-4}$ m/kN), where λ_I is equal to the relative shear displacement between adjacent stories (Fig. 3) (Ashkinadze *et al.*, 1990).

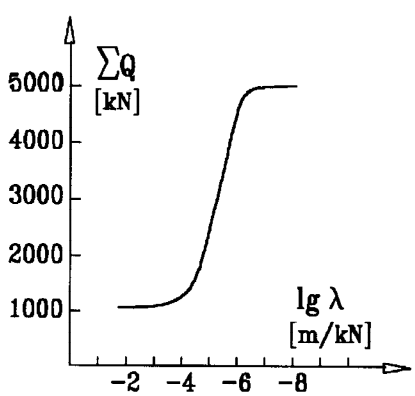

Fig. 3. Influence of vertical joints compliance on the seismic load for large panel buildings

Beyond this range the change of joint compliance has no influence on the structural stiffness of the system and the values of seismic loading, but might noticeably change the stresses (shear and normal) in joints and adjacent panels. From Fig. 3. it is evident that the design seismic forces are the function of joint rigidity. For the compliant (soft) joints between panels we have much lower seismic loads than for rigid ones. But despite the global reduction of seismic forces we might have substantial increase of stresses in separate bearing elements and a considerable increase in vertical reinforcement of panels.

The total lateral load on the building reached approximately 5700 kN which exceeded code seismic forces more than 3.5 times. However, this did not cause any significant damages to the main bearing elements. Several cracks appeared in lintels and joints of wall panels. The damage was mainly registered in non-bearing structural elements.

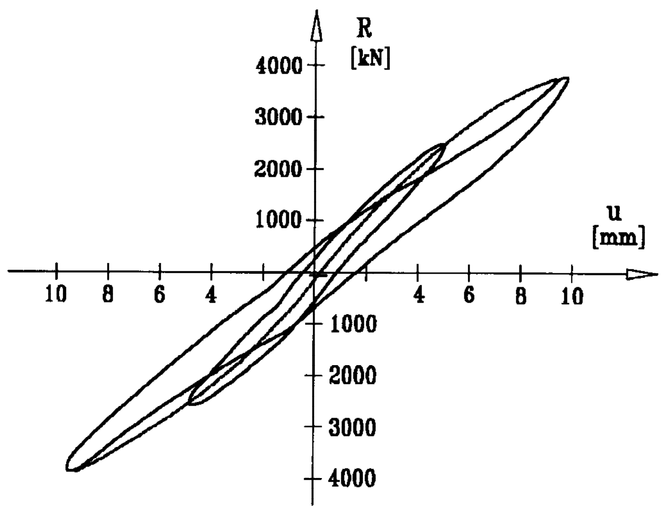

Fig. 4. Hysteresis diagram of the large panel building

The main characteristics of the large panel building under cyclic loading might be observed from the experimentally obtained hysteresis diagram shown in Fig. 4. It is evident from the narrowness of the hysteresis loops that the amount of the hysteretic energy dissipation is relatively small even for high lateral loading. On the other hand, it can be concluded that the capacity of large panel system to store elastic energy is much higher than for comparable reinforced frame bearing systems. This means that these structures exhibit less structural damage for moderate earthquakes, which is evident from the experimental results and the registration of the damages.

NUMERICAL ANALYSIS

Besides the research effort which was focused on experimental studies of large panel system, with emphasis placed on the behaviour of the individual wall and the coupled-wall system by means of the vertical and horizontal joints, the development of numerical procedures for the optimal design of these systems has also been conducted.

Numerical study of the experimentally tested residential building included both elastic and inelastic analysis by using versions of upgraded structural computer programs. The programs used were the upgraded ETABS (Wilson *et al.*, 1979) for elastic analysis and upgraded DRAIN 2D (Kanaan and Powell, 1973) for the purpose of inelastic analysis. The objective of the elastic analysis was to compare the measured initial dynamic characteristics of the test structure with the numerical predictions, and to explore the disagreement that exists between the various seismic codes for determining the seismic design forces for the large panel systems.

In order to achieve stiffness similarity between the experimentally tested building and numerical model several idealizations were considered. First model was most simplified by neglecting the effect of vertical key connections between bearing panels and by assuming the appearance of plastic hinges in connecting beams formed from lintel beams or floor slabs, or a combination of both. Therefore the vibration frequencies were obtained from the 3D elastic analysis assuming that the building is divided into a series of separated bearing panels, neglecting the stiffness of external fasade panels. Second model was developed by considering the partial strength effect of vertical connections and sufficiently stiff lintels. In that way 3D behaviour was obtained assuming that the building structure is divided into a series of separated wide column frames with flexible joints and members. The certain disagreement between the computed and experimental values (Table 1) for the first two models is due to the simplified idealization of the 3D model. In order to overcome this shortcoming third model was developed by assuming the real strength of vertical and horizontal connections between the panels. The 3D behaviour was also obtained by considering the interaction of perpendicular bordering internal and external wall panels. Table 1. shows the computed and in-situ experimentally measured vibration frequency periods for the first three modes.

Table 1. Results for the first three frequency periods

	model 1	model 2	model 3	experiment
T1	0.918219	0.289643	0.371692	0.27
T2	0.869448	0.25055	0.303873	—
T3	0.726957	0.190823	0.211677	—

The comparison between the tested structural displacements (excluding the foundation deformations) and the numerically computed displacements under various assumptions concerning the vertical joint compliance brought up very interesting conclusions about the stiffness of the system. It was registered that even under high intensity vibrations the real stiffness of the system corresponds to high rigidity of joints. Since the load reversal displacements of the vertical joints during the tests were relatively small (about 0.1 mm), the corresponding shear forces, Fig. 2. were also very small (approximately 10 kN per joint). This can be explained by high rigidity and shear strength of the horizontal joints and the interaction of the floor slabs. The deviations between the computed and the experimental values demanded the adjustment of the stiffness of all structural elements. This was done by assuming that the joint connections among the internal wall panels are rigid and by participation of the external facade panels in the global stiffness of the structure.

The 2D-inelastic dynamic analysis of the building was performed by computer program DRAIN-2D using the existing acceleration records. Neglecting the influence of torsional vibrations the analysis was performed for two orthogonal directions. The main purpose was to examine the inelastic deformations for obtaining adequate structural ductility and to examine the modification of forces and displacements in large panels due to the collapse of vertical and horizontal connections. In order to describe the flexural and shear behaviour of bearing panels a modified Al-Sulaimani-Roesett's model was adopted (Fig. 5) (Salatic ,1992).

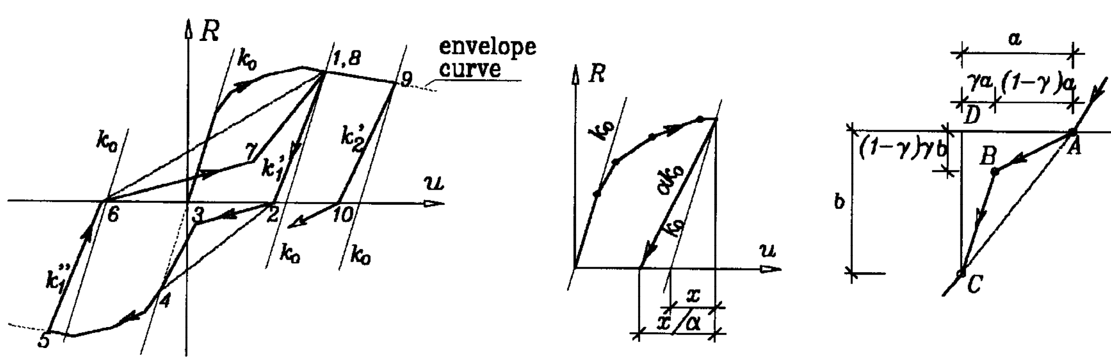

Fig. 5. Proposed hysteresis model

This model can involve degrading stiffness and strength with softening and pinching. The foregoing approach assumes that the hysteresis envelope is equal to the load-deformation curve imposed by monotonic loading, which was deduced from experimental investigations (Sekulovic, 1995). The experiment must provide the same conditions in specimen wall as in the part of real reinforced concrete wall. The main characteristics of the specimen are: geometry of wall including possible holes, reinforcement and vertical loading. The finite element model of the reinforced concrete wall subjected to the main part of El Centro (N-S component) earthquake lasting four seconds is presented in Fig. 6.

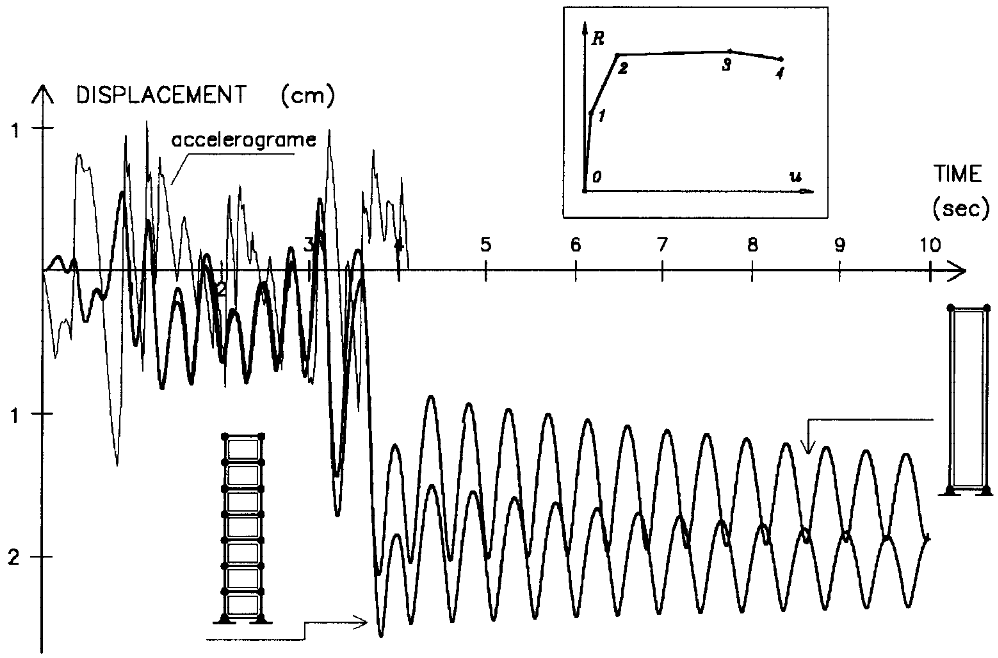

Fig. 6. Example of nonlinear panel behaviour

CONCLUSIONS

From the experimental and analytical research of the JINPROS large panel residential building the following conclusions may be drawn:

- The computed response of the building is in direct correlation with the distribution and the rigidity of vertical and horizontal joint connections. It might be concluded that the applied large panel system behaves as a dual system for earthquake loads. For moderate earthquake ground motions, bearing walls behave as "coupled" shear walls (by means of vertical and horizontal joint connections), and obtain very high levels of structural rigidity. Under strong earthquake loads bearing walls (considering the energy absorption capacity of vertical joint connections) behave almost as divided separate wall panels, developing quite ductile and stable hysteretic behaviour.
- Overall response of the structural system based on vertical joint connections between precast wall panels is quite reliable and insensitive to the type of earthquake input motion even with less ductility compared with other flexible bearing systems.
- Linear and nonlinear dynamic analysis indicated that current code distributions of equivalent static lateral forced are overestimated. Consequently, the required reinforcement can be reduced without changing the seismic stability of the building. The reduced amount of reinforcement might be approximately 30%.
- The satisfactory agreement between the inelastic numerically predicted and measured response values during high intensity vibrations were achieved by developing nonlinear panel finite element with certain physical and dynamical parameters deduced from experimental investigations.
- The analysis of in-situ experimental results showed that the ductile and hysteretic behaviour of specific building can not be considered without taking into account the soil-structure interaction.

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