SEISMIC EVALUATION OF PRECAST CONCRETE BUILDING WITH LARGE PANEL

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ABSTRACT

Precast concrete buildings with large panels, because of the speed of the construction and economic benefits are currently popular in the residential and commercial international construction markets. Horizontal and vertical connections are very important in these buildings and require special attention. Also, these connections are various and have the different response in an earthquake. However to understand the seismic behavior of the building versus input earthquake calculation demand parameter such as maximum inter storey drift and distribution of shear story is necessary.

In this study, seismic evaluation of prefabricated large panel buildings was undertaken. The research method is based on results from nonlinear dynamic analyses of the 3D model implemented with Opensees software to achieve this goal; prefabricated concrete large panel systems with different connection in 8 and 12 stories were designed. Interstory drift and distribution of the shear story in these buildings have been compared. At the end different risk levels have been studied. Results show that in taller construction with precast panel, it is better to use emulated joints.

INTRODUCTION

Lateral load resisting systems, divided into two main categories, Moment-resisting frames (with or without lateral bracing) and structural walls. Structural walls used in prefabricated buildings included prefabricated concrete load-bearing walls and prefabricated concrete non-load bearing wall. The large panel system (LP) is the structure has large prefabricated concrete load-bearing walls. The height of the walls is usually equal to the story’s height. These walls connect to each other with horizontal and vertical connections.

Building foundation’s Accidental motions during the earthquake induced forces within the building. Lateral load resisting system, spread these forces and transmitted them to the Earth. The system must be able to reduce the damage within acceptable limits and mainly guarantee the health of the residents. In Seismic zone, one of the principles of structural design is to lateral stiffness of structure is sufficient to control the transition between stories; this principle is to prevent injury of non-structural members in the buildings.

In small and medium-sized earthquakes, structural elements should be in the range of elastic, and have sufficient strength. In the event of earthquakes of high intensity the building must be ductile enough to withstand, high deformations (plastics) have. This bearing must prevent the collapsed. Accordingly, designers are simultaneously considered lateral force resist, rigidity, strength and energy absorption capability in the design of structures.

Large Panel buildings with respect to the placement of the panels together, three distinct are divided. 1-Transverse wall systems (which are load bearing walls in width), 2-The longitudinal walls (the walls are load bearing for long only) and 3- Two-way system (which walls are load-bearing longitudinal and transverse walls).

Connections have an important role in LP buildings. Because they affect the speed of installation panels together and create the structural integrity. Precast concrete panel can be classified into two types: jointed and monolithic. In monolithic construction the precast concrete panels are joined by connections which possess stiffness, strength and ductility comparable to those of the precast concrete panels, and the design emulates the characteristics of a cast-in-place concrete structure. In contrast, jointed structures contain connection details which have significantly lower stiffness and strength properties than the precast concrete panels, and when the elastic limit of the structure is exceeded, ductility demand is concentrated in the connections between adjacent precast concrete panels (Wilson et al 2007).

In jointed constructions because of the inherently low stiffness of the drypack material in comparison to the concrete typically used for precast wall panels, it was observed that the top wall panel rotates in rigid body motion with respect to the bottom panel. All the deformation was concentrated in the joint region and, therefore, shear deformations and flexural within the panels could be neglected. Total deformation of the top fiber of upper wall panel, Δ, could be expressed as a combination of slip deformation, Δs, and rocking deformation, Δr, as shown in Fig. 1:

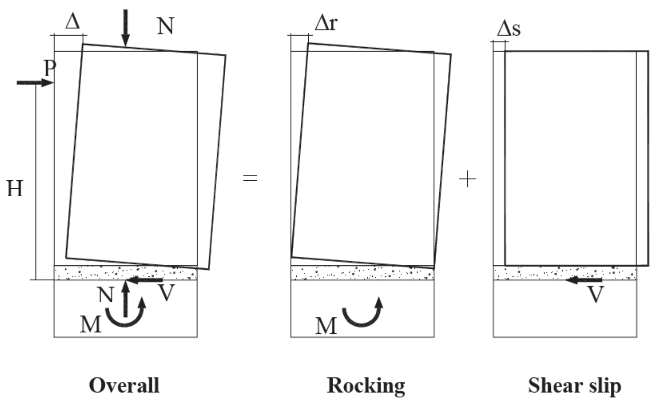


Fig. 1. Components of deformation: (a) rocking; (b) slip.(2) (Hashemi 2011)

When earthquake force is applied to LP structures The shear failure mechanism is capable of dissipating the energy exerted on the structure and large forces or large displacements not transferred to the structure. Nevertheless Llorente (1979) found that shear slip may be undesirable because resistance depends largely on friction and when sliding starts it is liable to lead to accumulated unrestrained displacements under certain earthquakes when the slip occurs predominantly in one direction. There is certain danger in having unrestrained displacement because large secondary (P-Delta) moments develop and eccentricity will occur in perpendicular walls.

A rocking mechanism dissipates little energy and creates severe force concentration in the compression region. As the wall rocks open, all the axial load and the compression force of the flexural couple has to be resisted in a small compression stress zone at one end of the wall. Compression crushing of the concrete may occur, leading to instability. Rocking's main advantage is that it should not result in accumulated displacements.

Several experimental investigations on large panel wall systems have been reported in recent years.

Becker et al.'s (1979) have done studies on the seismic response of the concrete buildings with prefabricated concrete wall panels, they the model of the two buildings with 10 and 5 storey on the nonlinear inelastic analysis of structures using finite element method to evaluate the response to the force of the earthquake. In this paper they develop a computational model suitable the general behavior of horizontal connections in the behavior of large precast panels a crater under different parameters were investigated. In this model it is assumed that all non-linear and inelastic behavior occurs wall connection and Wall and thus remains in the elastic range. Connections are modeled as boundary elements and the ability to model that the deflection of the wall and the wall displacement due to slip to consider. Becker has admitted a structural rotation deformations is just wasted energy in buildings is minimal. While sliding walls and floors would limit the energy dissipation structures to earthquake forces. Rocking deformation of the wall will have a significant impact on the structural base period. Performance of both Rocking and sliding deformation causes a significant force in the corner of the wall and bottom of the Connecting focus. If the area of the connection is not designed for such forces reduce the optimal performance of large structures is expected earthquakes.

Olivia et al.'s (1990) studies in experimental models of wall panels with concrete frames. In this study, a series of seismic tests on wall panels as part of a research program between the United States and Yugoslavia has been done. The study examined a sample prefabricated wall, quasi-static (cyclic deformation) similar samples were shaking table test panel. The results showed that the overall structural response mechanisms are the same in both experiments. Along with increasing rotational deformation, damage caused by tension and pressure at both ends of the connection between the panel expands. This expansion will give cracking, buckling and ultimate failure of vertical reinforcement horizontal connections. Of course, there are some differences between the two models. First, of course, important differences-amount of energy dissipated in the static tests have been much more of shaking table tests. Quasi-static tests unrealistic amount of wasted energy to produce a dynamic response. For slow loading of the test – to shaking table tests - more of a non-linear material behavior and become more energy waste. Another difference in treatment failure asymmetrical wall panel is shaking table test, if the test quasi-static displacement control on the wall due to the failure of the asymmetric extend and maintain. Olivia et al.'s stated although static load test is a good way to learn about the behavior of a structure. However, it should be tested to determine the exact behavior of a vibrating table or similar numerical analysis according to specifications obtained from static tests done.

Soudki et al.'s (1995) have been done studied on horizontal conection for precast concrete shear walls. They conducted full- Scale specimens test to investigate the behavior of mild steel connections for precast concrete shear wall panels. The spicmens in their study were subjected to reversed cyclic combined flexure and shear in addition to constant axial stresses normal to the connection. They discussed the influence of cyclic vs. static loading, mechanical splicing vs. welding of reinforcement, and mechanical splicing vs. bolting of reinforcement to a tube section. They also presented effects of the use of shear keys and partial debonding of reinforcement on the behavior of the connection. they developed a simple analytical procedure to predict the envelope of the cyclic response.

Anderson et al.'s (2004) examined precast concrete panels with openings. This is an example of the cavities in the wall panels for doors and windows that have been placed under the cyclic loading test was And seismic characteristics were studied. The results showed that these walls show a high strength and stiffness but it is low formability. Also the walls are cracking down panel also examined Suggestions for reducing the cracking were presented with respect to the reinforcement setup. In 2006 Pekao et al.'s the progressive collapse of prefabricated structures with large panels surveyed. They have modeled structures with discrete element method And for the case of failure or damage caused by the earthquake were examined progressive. At the end of the vertical joints to design appropriate safety margin provided.

Pekau et al (2006) have done studied on progressive collapse in this structures. they used discrete element method in their modelling and studied the mode of failure and damge in the earthquake. At the end they provide appropriate methods with safety margin for modelling the vertical joints.

Wilson et al (2008) have done evaluated jointed precast concrete load-bearing panel structures in areas with low to moderate seismicity. They also carried out sub assemblage tests on the commection between the wall panel and floor slab in order to calibrate finite element model for typical load displacement characteristics.

In this study, the nonlinear dynamic analysis of structures with prefabricated concrete panels. Plan on Floors 8 and 12 paid and distributed maximum shear of storey in two risk level DBE and MCE compared.
In this study the envelope of the cyclic response of the connection that suodki (1995) presented is used in modelling of the connection.

STRUCTURAL SYSTEM

The selected structure is an residential building that is constructed with typical plan for residential buildings (Figure 2). The structure is 36 m by 18.4 m in plan and each story‘s height is 3 m. The model primary analysis and design within ETABS V9.7.0 software accordance ACI 318-08 code. The precast concrete load-bearing panels constructed from from material that shows in table 1. The panel thickness is 150 mm for 8 story and 200 mm for 12 story structures. They are reinforced with mesh consists of 8 mm and 12 mm diameter bars at 200 mm spacing for 8 story and 12 story respectively with a yield stress of 400 MPa. placed centrally with 40 mm cover to the outside face of the panel. The suspended floor slab is a 150 mm hollow core one-way slab with a structural topping that spans the shorter dimension of the structure. Table 2 shows the dead and live load that apply to the floors.

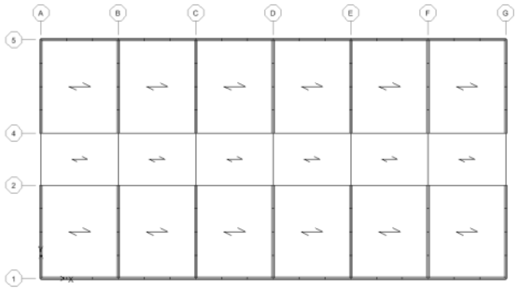


Figure 2. plan of the buildings

Table 1. Properties of building material

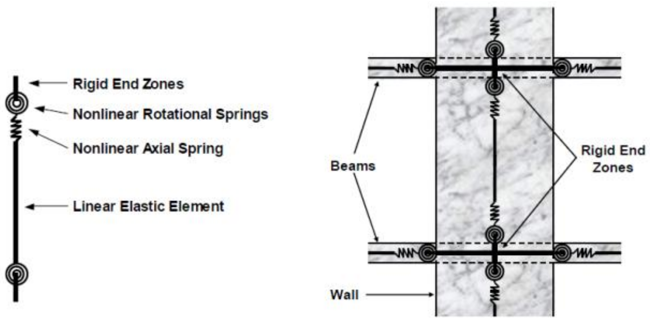
Structural elements	F'c(Mpa)	Ec(Gpa)	Fy(Mpa)	Es(Gpa)
Wall and concrete slab	27	25	400	210

Table 2.Building Loading

Level	Dead Load (Kg/m)	Live Load (Kg/m)
Roof	300	150
Floors	350	200

MODELLING PRECAST CONCRETE BUILDING WITH LARGE PANELS

A: Wall panel modelling, modeled with concentrated plasticity model (John Wallace, 2006). In this modelling, the wall model with an elastic column in a central location of the shear wall, and there are non-linear springs at the end of the column. Fig3. Two side walls connected by a rigid beams. FEMA 356 parameters for precast wall are used For modelling springs inside prefabricated panels.



(a) Beam-column element (b) Model configuration

Figure3. Modelling wall shear beam column element method (John Wallace, 2006)

B: Connection modelling;

All inelasticity would take place in the joint region. Two approaches have been used to model this connection (1) as a continuous media, with multi node inelastic rectangular contact or interface finite elements, or (2) with discrete inelastic no dimensional spring elements (Clough et al 1989). In this study, springs are used to model the connection between large panels. In horizontal connection slip and rocking created because of the shear and the torsion in this region, so shear springs and rotational spring show this

behavior. In vertical connection slip and pressure created because of the shear and normal forces. The spring's parameters obtained from the previews studies(Fig 6).
Two different types of Horizontal connections are compared in this study. First is the connection that Soudki and colleagues (1995-1996) earned the hysteresis behavior from the test. Reinforcing bars spliced by a sleeve - This connection is typically used by the American and Japanese precast concrete industries. The continuity bars are connected by NMB splice sleeves, as shown in Fig. 4. The protruded straight bar from the top panel is placed inside the splice sleeve, which is embedded in the lower panel. The connection is dry packed and the splice sleeves are pressure grouted with a non-shrink, high strength, SS mortarReinforcement attached to the sleeve in this connection. Second connection is emulated connection, that model with rigid connection. Emulated building has an integrated behavior, and in this case, plastic hinges made in the wall or in a place away from the connections. Vertical joints between walls are wet joints. According to research by Kianosh et al (1986) this vertical connections have rigid behavior.

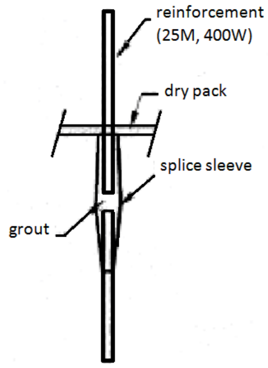


Figure4. Connection RS detail

3D model of precast concrete with large panel buildings implemented with Opensees software. These Model assumptions are: 1- The interaction between soil and structure is not considered. Connection of the structures to the land is rigid. 2-Floor diaphragm is rigid. 3- The connections between the diaphragms and the panels have elastic behavior.4- To simplify regardless of the opening. To validation this modelling, one of the structures that pekau et al (1991) analysed was modelled with above approach, and results was compare. As seen as figure 5 the result is near and this show that method of the modelling make correct answers.

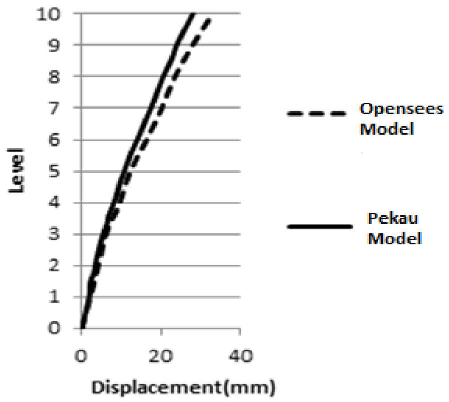


Figure5. The validation of the model in Opensees (maximum displacement changes in altitude)

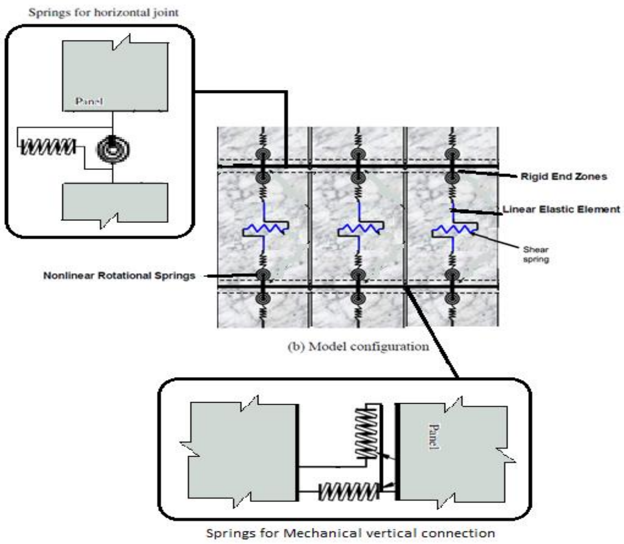


Figure 6, Horizontal and vertical connection between panels

DYNAMIC ANALYSIS

The purpose of nonlinear dynamic analysis is to study the behavior of structures during earthquakes, considering the nonlinear behavior of the members, determination the desired performance level and accurate Accelerogram frequency. In nonlinear dynamic analysis, structural response determined with considering nonlinear behavior of materials and geometrics. In this method, it is assumed that the damping matrix and stiffness matrix changed in each step but they are constant in during one step. The response of structures under occurred real Accelerogram using numerical methods and is calculated for each time step.

The structural analysis in each direction should carried out with at least three pairs Accelerogram. For dynamic nonlinear analysis, the seismic forces in two perpendicular directions enter to the 3D model at the same time. The effect of the vertical component is ignored, Because of lack of pre-tensioned or long cantilever beam in the building. If less than 7 accelerogram be considered in every direction, the maximum structural response should be considered responses. Three pairs accelerogram used in Table 3 are observed.

Maximum story shear distribution curves for the two levels of risk for MCE and DBE construction of both a height of 8 and 12 floors can be seen in Figures 5 and 6. In this study as said as above to LP buildings are compared. The Model 1 is jointed structure that during the analysis, precast concrete panels remain in the linear elastic range and nonlinear behavior is in the horizontal connections. The Model 2 is emulated LP building, which connections are rigid, and inelastic behavior occurs in the panels.

CONCLUSIONS

Time history analysis of three pairs Accelerogram used. The final structure is reflected in every moment of time equal to the maximum reflection is obtained from the analysis of these three couples Accelerogram.

Changes almost linearly with shear profiles representing the first mode is the rule. Cutting-year increase in 2475 compared to 475-year level of risk can be seen as well.

1- Horizontal connections in this study are from modeling the behavior of laboratory samples that researchers in the current experiments, which were obtained by trial and error.

2-The horizontal joints between the panels model by two springs, first is for modeling the shear and the second is for modeling the bending. In jointed building properties of the springs is nonlinear, and in emulated buildings the connections become rigid.

3- The primary stiffness of the building is reduced by increasing the number of stories and the lateral load capacity can be increased.

4-Emulated concrete prefabricated buildings 8 and 12 at the functional level of life safety and Collapse Prevention may not be collapse. Maximum interstory drift in the life safety performance level is less than 1%

and lower than 2% in the Collapse Prevention performance level. In general it can be concluded; in taller building should use emulated connection.

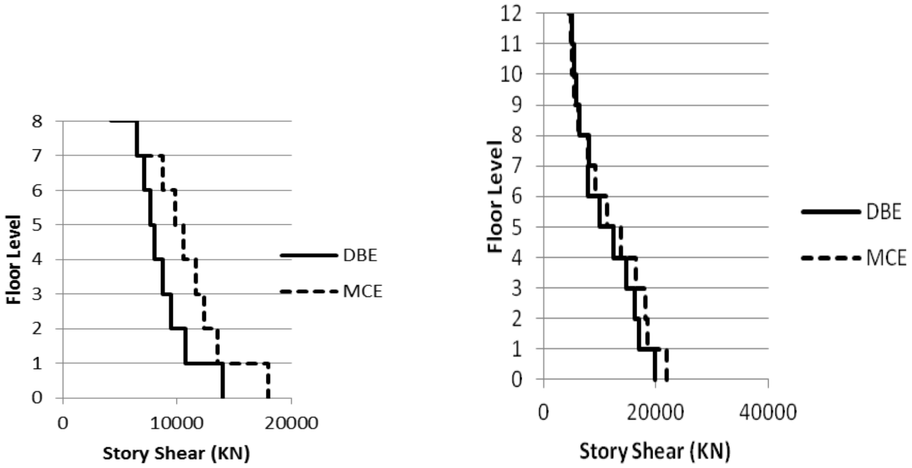


Fig7. Distribution of maximum shear story for two risk level MCE and DBE (8 and 12 storey, Model1)

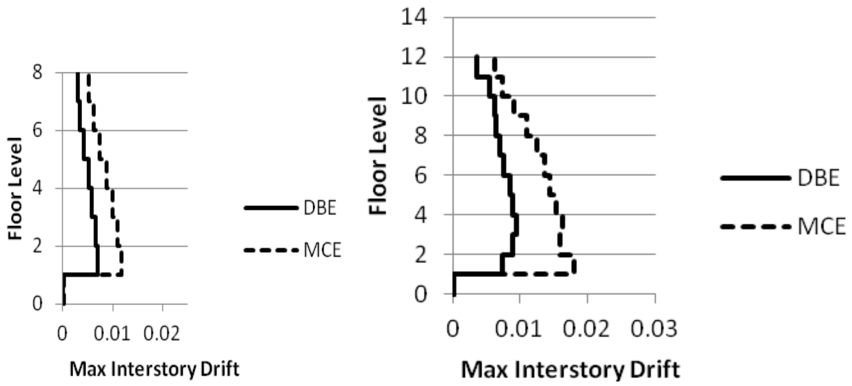


Figure8.Distribution of maximum drift for two risk level MCE and DBE(8 and 12 storey, Model1)

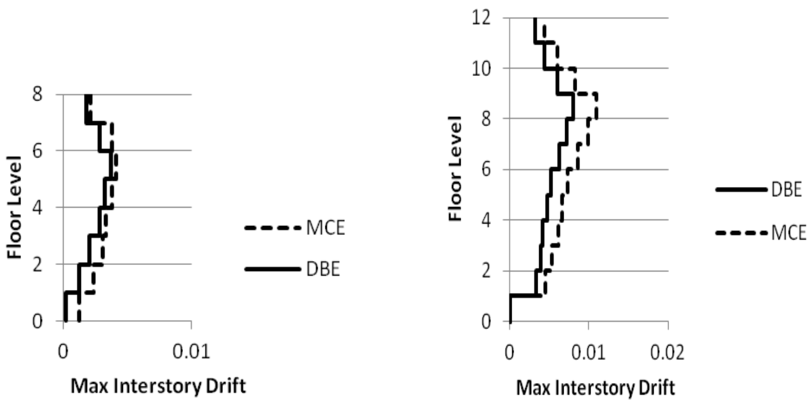


Figure9.Distribution of maximum drift for two risk level MCE and DBE (8 and 12 storey.Model2)

Max Interstory drift	سطح خطر	تعداد طبقه	مدل
0.007	DBE	8	Y Direction مدل ۱
	OK		
0.0111	MCE		
	OK		
0.003	DBE	12	
	OK		
0.0043	MCE		
	Not OK		

Max Interstory drift	سطح خطر	تعداد طبقه	مدل
0.007	DBE	8	Y Direction مدل ۲
	OK		
0.0111	MCE		
	OK		
0.0082	DBE	12	
	OK		
0.012	MCE		
	OK		

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