Evaluación y refuerzo sísmico de estructuras de paneles prefabricados mediante análisis pushover

Seismic assessment and retrofit of precast panel structures using pushover analysis

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RESUMEN

En la ponencia se presentan una serie de análisis no lineales mediante el método pushover según la ASCE41-13, que se utilizaron para la evaluación y refuerzo sísmico de varias estructuras de paneles prefabricados. Para abordar de forma realista su complejo comportamiento, se realizaron estudios previos de los conectores y paneles, los cuales presentaban un comportamiento muy frágil, debido a la presencia de bajas cuantías de armado. Finalmente, se presentan ejemplos de refuerzo mediante aplicación de fibra de carbono (FRP), unión mediante conectores metálicos y hormigonado de una capa in situ en forjados.

ABSTRACT

The paper presents some results from nonlinear pushover analysis used to assess the seismic performance of precast panel structures according to ASCE·41-13. Critical aspects related to modelling in-plane panel failure, connections and interaction between elements are discussed based on past projects in Israel. Finally, examples of seismic retrofitting adopted for enhancing displacement capacity and energy dissipation using FRP, steel connectors and in-situ concrete topping are presented.

PALABRAS CLAVE: estructuras paneles prefabricados, evaluación sísmica, refuerzo, pushover.

KEYWORDS: precast panel structures, seismic assessment, retrofitting, pushover

1. Introduction

Precast concrete structures present a number of advantages with respect to cast-in-place solutions, such as speed of construction, quality control and cost effectiveness. However, the lack of adequate load path continuity can result in a catastrophic seismic performance as demonstrated in past earthquakes (1988 Armenia [1], 1994 Northridge and 1999 Kocaelli, Bucharest, Rumania 1977, 2012 Emilia-Romagna, Italy). Commonly observed failures involve failure of connections, beams and slabs dropping off the supports, joint opening and pounding and displacement incompatibility causing tearing of floor diaphragms among other issues [2, 3].

The paper summarizes recent investigations on the seismic performance and retrofitting of precast panel buildings in Israel. Several buildings of this type were built for residential and public use in regions of moderate to high seismicity (PGA ≈ 0.1g-0.4g). After a brief description of the typical prototype building, modeling considerations for nonlinear analysis with emphasis on panel behavior and connections are presented. Results corroborate the expected poor performance of these structures, which usually collapse due to sliding and connection failure at very low lateral drifts. An overview of implemented retrofitting measures is finally discussed.

2. Prototype buildings

The typical building consists of a mid-rise structure with multiple bays constructed with precast wall panels and hollow-core or flat slabs (Fig.1). Panel thickness varies between 150-200mm depending on the level of axial load and position of the panel (interior or exterior). Panels feature openings for doors and windows and sometimes embedded insulation. Panels with no openings have a minimum amount of reinforcement consisting of a double layer mesh of ϕ4@15/15. Panels with openings have reinforcement around the opening designed for gravity loads (Fig.1). The connection between panels in the vertical direction typically consists of starter bars in the bottom panel and inserts in the top one, embedded within rectangular pockets at two locations. The connection between panels in the horizontal direction consists of pairs of bolts protruded through the panel and connected to a vertical bar with a welded ring. Usually, concrete is poured within the empty space left in between. Some panels, such as the 150mm thick panels used for partitions, do not present any bolts in the horizontal direction but only concrete filling. In some cases, precast panels are combined with cast in-situ frames and shear walls. These walls are intended for basement shelter or staircase shafts, but not for seismic purposes. Field investigations identified very poor concrete material in these walls, known as Debesh, with compressive strengths of ≈20MPa and very low reinforcement.

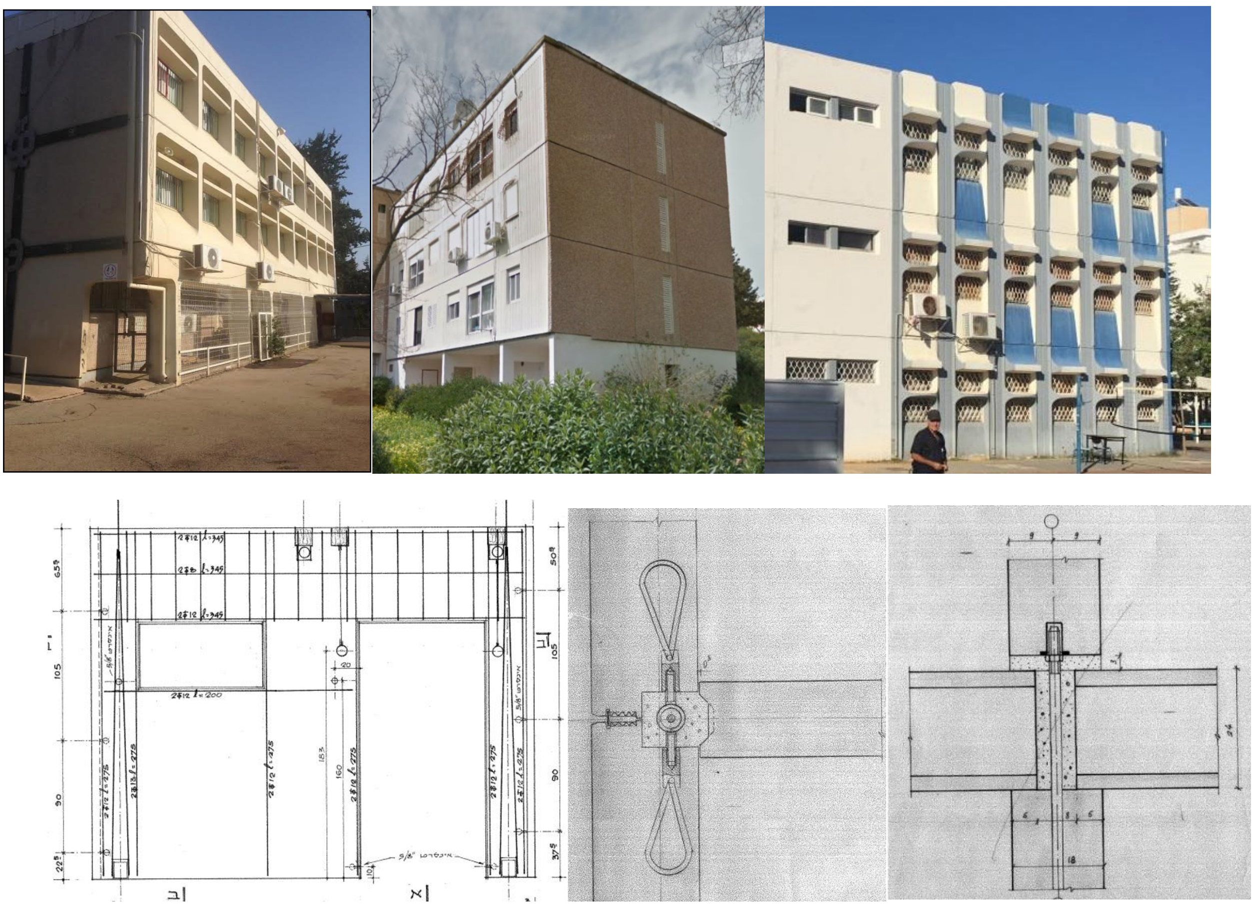


Figure 1. Typical precast panel buildings in Israel and detailing of panels and connections.

3. FEA of precast panels

A set of nonlinear FEA analysis were undertaken on panels with different geometric and reinforcement configurations to assess their performance under lateral loading. Fig.2 and Fig.3 exemplifies the response of a panel with openings subjected to different levels of constant axial load ratio (between 10-25%). The maximum capacity reached almost 500kN at 2mm lateral displacement, shortly after which softening occurred. The lateral capacity is significantly influenced by the amount of openings. For a larger area of openings, the response is more similar to that of a frame, i.e. more ductile but failing at lower lateral strength. This is shown in Fig.3 where the lateral capacity, normalized by the cross-section area, is plotted as a function of the percentage of openings with respect to the total area of the panel. The data includes panels with different reinforcement ratios and axial loads which increases the scatter, however there is a clear trend in the capacity reduction due to the presence of openings.

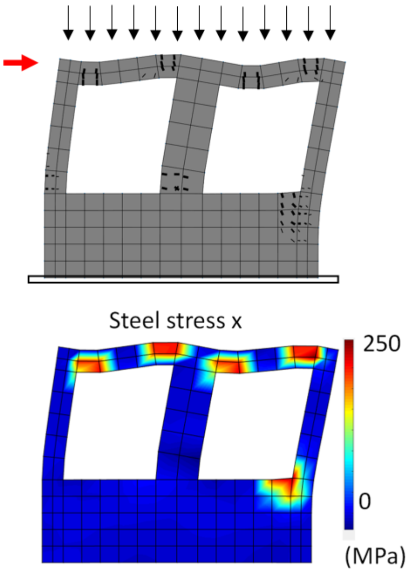


Figure 2. FE Analysis of individual panel: crack patterns and x-reinforcement stresses.

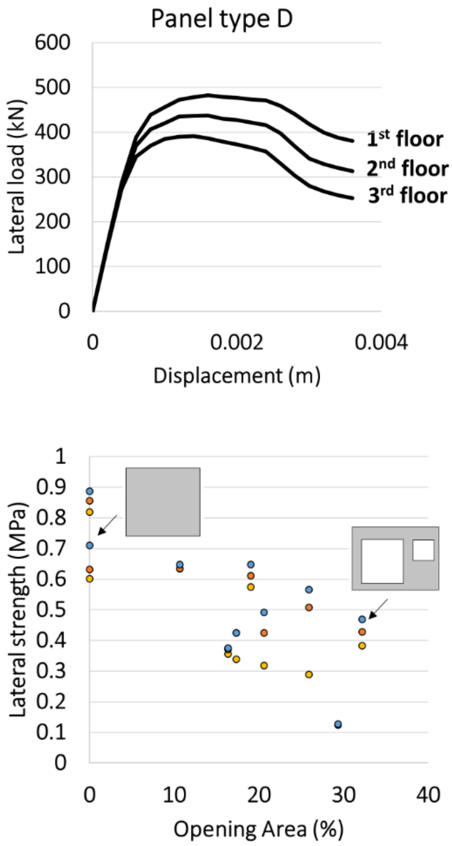


Figure 3. Lateral panel response for different axial loads (*top*), lateral strength as a function of openings (*bottom*).

The exact boundary and loading conditions of the panels within the actual building is not straightforward. Assuming the bottom of the panel fully fixed and the lateral load distributed along the top will result in flexural behavior and horizontal cracking. However, diagonal cracking was observed in some existing panels (Fig.4), probably due to interface shear stresses along vertical and horizontal joints. Fig.4 shows results from nonlinear analysis considering these shear stresses combined with axial compression. A different load-transfer mechanism is activated, characterized by the formation of a diagonal compression strut and tensile cracking. This agreed well with the crack patterns observed in one of the precast buildings which had suffered a differential settlement of 12cm over a length of 22m. It indicated however a very brittle post-peak response after reaching lateral capacities as high as 1300kN.

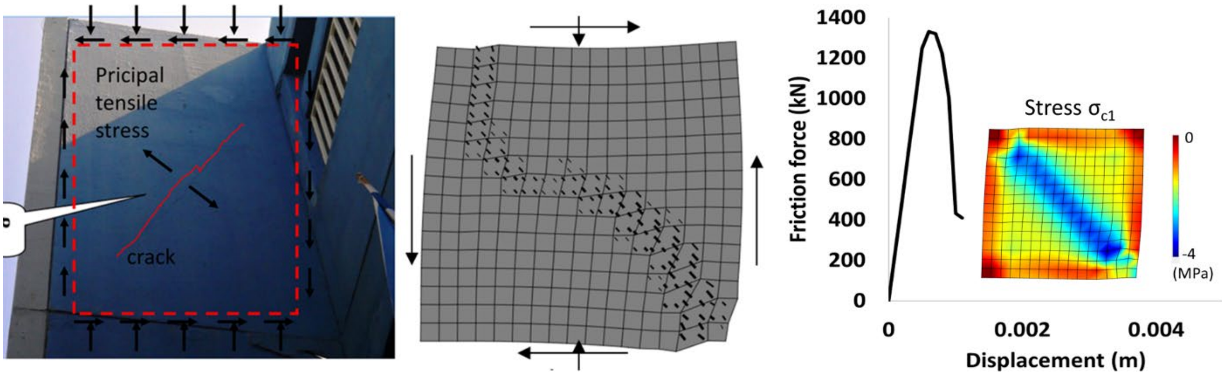


Figure 4. Observed cracking in one of the exterior panels and corresponding FEA results.

4. Global Analysis

Global models of the buildings were defined for pushover assessment taking into account the response of panels, connections, diaphragms, etc. and their interaction as explained below. In general, a combination of 2D and 3D models was used. 2D models were found useful for preliminary assessment and more practical in terms of defining panel connections. For large 3D models defining the in-plane and out-of-plane connectivity for each panel may require too much time and easily result in numerical problems. 2D analyses were used to identify key aspects in the collapse progression which allowed introducing some practical simplifications in the 3D models.

4.1 Modeling panels and connections

Panels were modeled with an equivalent horizontal spring representing the lateral force-displacement response obtained for FEM analysis. The backbone curve was defined by the secant stiffness, maximum lateral force and ultimate displacement, after which the capacity drops to zero. The horizontal spring was connected between the top and bottom nodes of the rectangular panel consisting of rigid horizontal elements and vertical springs representing the axial stiffness (Fig. 5). The panels were connected to the slab and other panels by means of axial and shear springs representing friction, contact and resistance of bolts and dowels in tension and shear. Friction was characterized with a bilinear force-displacement response with a very high initial stiffness and the maximum friction force obtained as a product of the friction coefficient between 0.40-0.60 and the axial load at the corresponding level. The actual panels are connected vertically with starter bars in the bottom panel and inserts in the top panel, the vertical separation being about 25cm. The empty space is filled with plain concrete. The dowel action provided by the starter bars acts in parallel with the friction, which has a very high initial stiffness, hence it will become effective only after some amount of sliding has occurred. The lateral force at yielding of the vertical bars was estimated as 4kN, which contribution to the global base shear is negligible. The vertical resistance in tension is provided by pulling of the bar, and in compression by contact between concrete surfaces. This results in an asymmetric force-displacement curve as shown in Fig.5. At the vertical joints, precast panels were connected to each other by bolts inserted in the panel and

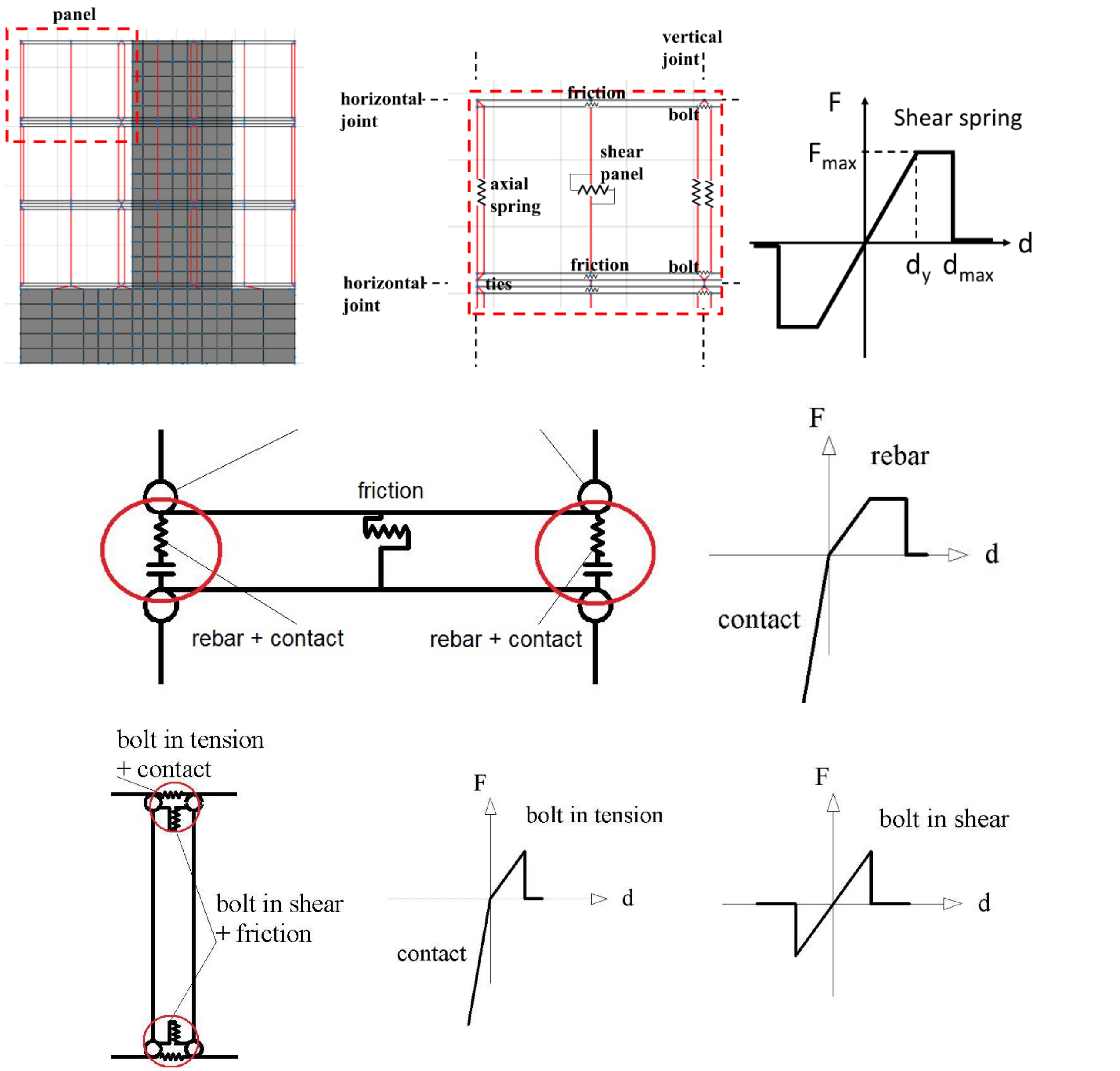


Figure 5. Modeling panels and their connectivity for global analysis.

connected to a vertical reinforcement bar with a ring welded to the head of the bolt. The space left between panels was filled with concrete. There were three ¾" diameter pairs of bolts located at each side along the height of the panel. The tensile and shear resistance of the bolts was modeled with horizontal and vertical spring elements (Fig.5). Since each pair of bolts act in series, both in axial and shear directions, the equivalent stiffness is given as $K_{eq}=(1/K_1+1/K_2)^{-1}$, where K_1 and K_2 is the axial (or shear) stiffness of bolt 1 and 2, respectively.

4.2 Pushover Analysis

Permanent and incremental loads need to be defined in the global model for pushover analysis. Permanent loads corresponding to self-weight, dead-load and 30% of the live-load were distributed over the diaphragms accounting for p-delta and axial-flexure interaction effects. For relatively stiff mid-rise buildings, as in the present case, second-order bending moments due to p-delta have little influence, except perhaps in the post-peak range at large displacements where the structure has already failed. Regarding incremental loads, these represent equivalent forces which distribution along the height reflects the inertial forces imposed during seismic excitation. For first-mode dominated buildings this distribution can be taken proportional to the 1st elastic mode shape. Thus the vector of lateral forces is given as:

$$\boldsymbol{F} = \lambda \boldsymbol{M} \boldsymbol{\phi} \tag{1}$$

where λ is the load factor, **M** is the mass matrix and **Φ** the 1st mode displacement shape. After computing the capacity curve, the performance point is calculated according to ASCE41-13 [4] as:

$$\delta_i = C_o C_1 C_2 S_a g T_e^2 / 4\pi^2 \tag{2}$$

where:

C_0: Mass participation factor x ordinate of the first mode shape

C_1: is less than the spectral value at T=0.2s for T<0.2s

is =1 for T>1s

is $=1+(\mu_{strength}-1)/aT_e^2$ for 0.2s<T<1s

$\mu_{strength}$: force reduction factor $=S_a C_m W/V_y$

C_2: is =1 for T>0.7s

is $= 1+1/800((\mu_{strength}-1)/T_e)^2$ for T<0.7s

S_a: Spectral acceleration at the T_e

T_e: Effective Period, given as: $= T_i(K_i/K_e)^{0.5}$

where K_i is the elastic stiffness, T_i the elastic period and K_e the effective period determined from the idealized pushover curve. The target displacement corresponds to the control node of the multi-degree-of-freedom system. It is obtained based on the trilinear idealization of the MDOF pushover curve.

Fig.6 exemplifies results from 2D pushover analysis of a 3-story precast building, showing base shear-top displacement response and corresponding deformed shapes at different time steps. The structure responds nearly elastic up to 2.6mm displacement, after which stiffness reduction occurs due to horizontal sliding between wall panels. The maximum base shear was 1500kN. Most of the sliding concentrates at the top story due to lower axial loads and hence lower friction forces. At 6.4mm displacement, the top bolt of the last panel at the 2nd level fractures producing brittle global collapse. The target displacement according to ASCE 41-13 at 10% probability of exceedance in 50years was estimated as 13mm, which exceeds the displacement capacity. Hence the structure was seismically retrofitted as explained below.

Fig.7 exemplifies results from 3D pushover analysis of a 4-story building with precast panels, frames and concrete walls with rectangular and U-shaped walls. The response is asymmetric, depending on the loading direction. In the positive direction, the U-shaped wall presents higher capacity due to the tensile contribution of the flange region. In the negative direction, tension forces in the U-shaped wall are carried by the web regions which have lower tensile capacity. The response was dominated by failure of the U-shaped wall and sliding of the panels. The maximum capacity was 600kN (positive direction) and 800kN (negative direction). Flexural cracks in the wall were observed in the

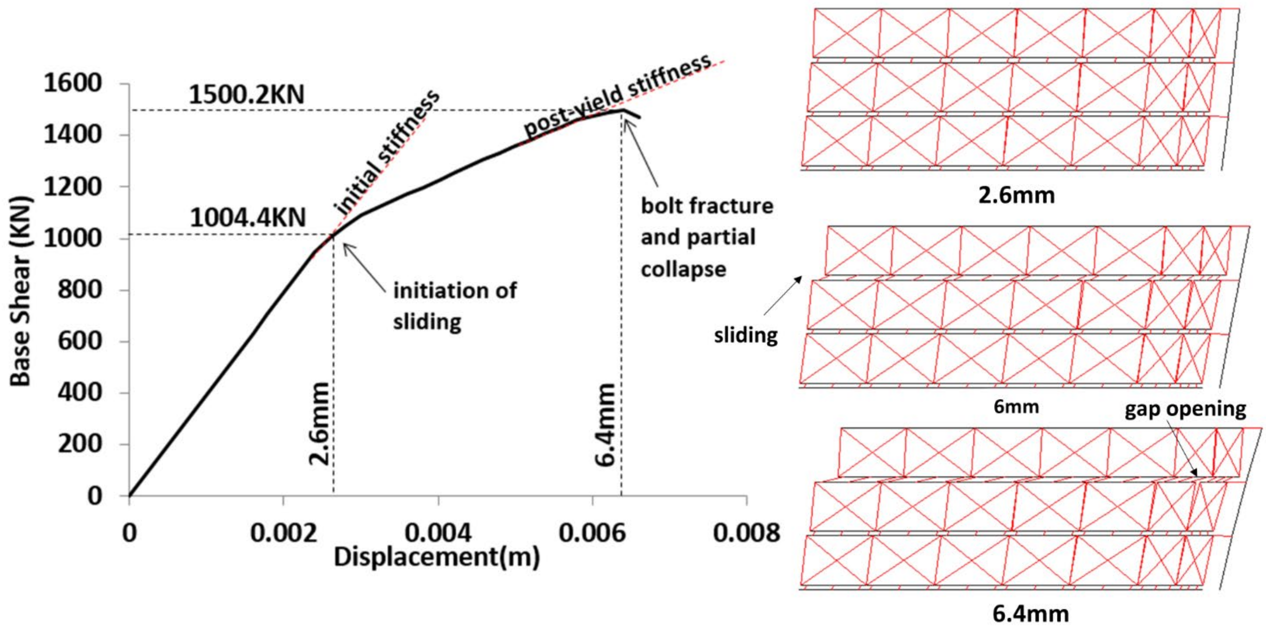


Figure 6. Lateral force-displacement response for a 2D model and displaced shapes at different levels.

negative direction. The slabs were modeled as flexible diaphragms accounting for the lack of concrete topping on the beams. Global failure was assumed at initiation of sliding after wall failure. Although sliding and friction can be effectively used for energy dissipation, in the present case sliding is rather "uncontrolled" and cannot be considered as perfect sliding. In the actual building sliding is expected to produce kinematic incompatibilities between elements due to construction tolerances, kinematic constraints, misalignment, settlements, etc.

5. Seismic retrofitting

The following table summarizes some of the retrofitting measures adopted in the previous projects to improve the seismic performance:.

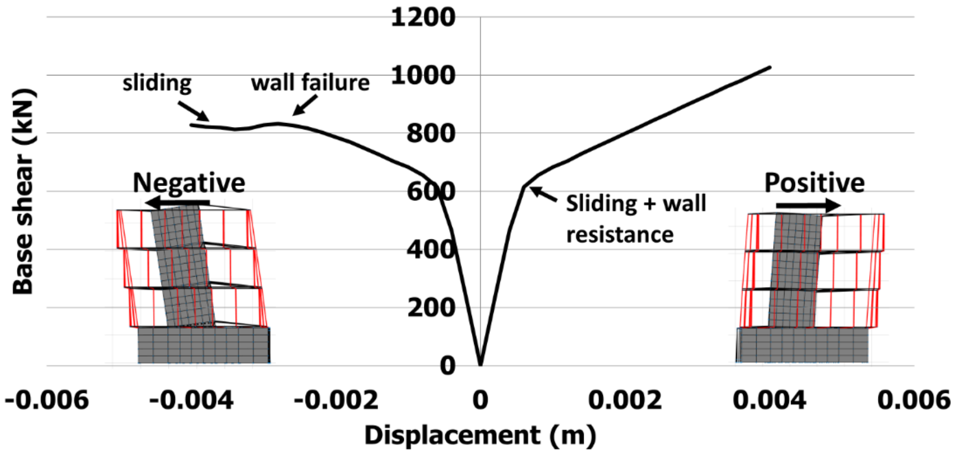
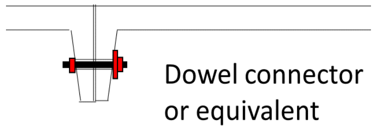


Figure 7. Lateral force displacement response for a 3D model in positive and negative directions.

Table 1. Seismic retroffiting schemes.

<table>
<tr><th>Intervention</th><th>Objective</th><th>Schematic Detail</th></tr>
<tr><td>Connection of precast floor slabs or casting of a concrete top layer (topping)</td><td>Achieve rigid diaphragm action, load path continuity, avoid pounding.</td><td>
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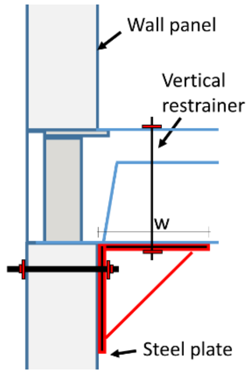
<tr><td>Increasing seat width of slabs</td><td>Load path continuity, provide support</td><td>
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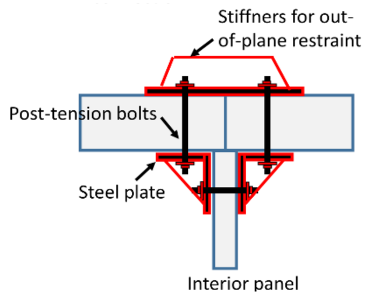
<tr><td>Strengthening wall panel connections</td><td>Increase lateral strength, avoid connection failure and response irregularity, provide load path continuity</td><td>
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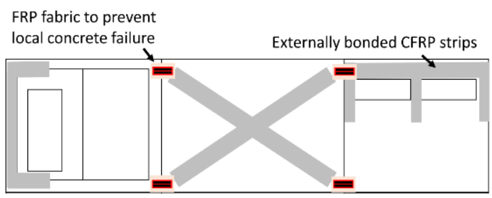
<tr><td>CFRP wrapping of wall panels</td><td>Increase panel shear strength, provide confinement and ductility, increase total base shear</td><td>
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6. Conclusions

Modeling considerations for nonlinear seismic analysis of precast panel buildings have been presented. Main focus was placed on panel failure, connection and interaction between panels including sliding and joint contact. Numerical results corroborated the poor seismic performance of these structures, with displacement capacities in the order of a couple of millimeters. Selected retrofitting measures for strengthening panels, connections and diaphragms were finally summarized.

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