Quasi-Static Loading Strategy for Earthquake Simulation on Precast RC Shear Walls

István DEMETER, Tamás NAGY-GYÖRGY, Valeriu STOIAN, Daniel DAN
Department of Civil Engineering
Politehnica University of Timișoara
T. Lalescu 2, 300223 Timișoara
ROMANIA

Abstract: - The loading strategy presented hereinafter, was conceived to reproduce the seismic action, on precast reinforced concrete structural walls, in laboratory conditions. The experimental wall panels were constructed with original or cut-out, narrow or wide door openings, respectively without opening (i.e. solid wall), and will be strengthened with FRP composites in prior-to-damage or post-damage state. The large number of experimental variables caused difficulties regarding the applicable vertical and lateral loads. Furthermore, the different experimental elements were expected to have altering concrete quality, condition which could not be disregarded in order to obtain consistent results. According to the currently established test methods for quasi-static earthquake simulation, the vertical loads were assigned employing the normalized axial stress parameter and a displacement controlled cyclic lateral loading history was designated.

Key-Words: - RC wall, normalized axial stress, cyclic loading history, displacement control, precast large panel, cut-out opening, experimental test, seismic action.

1 Introduction
This paper is a preparatory study of the experimental research concerned with the seismic behaviour of the Precast Reinforced Concrete Wall Panels (PRCWP) weakened by cut-out openings and strengthened with Fiber Reinforced Polymer (FRP) composite materials. A series of eight, 1:1.2 scale wall specimens (Fig. 1) were constructed, with variable opening configuration and reinforcement arrangement. The experimental elements will be subjected to in-plane reversed cyclic lateral (horizontal) and pseudo-constant axial (vertical) forces, simulating the seismic loading conditions at a quasi-static rate. In this study the authors aimed to determine the loading strategy, in accordance with the requirements of the currently established experimental methods for earthquake simulation, taking into consideration, at the same time, the available testing equipment limitations.

2 Literature Review
Extensive experimental research was performed by several authors related to RC walls subjected to in-plane seismic loading conditions. Regarding the applied vertical loads, various cases were identified: in research work reported in [1, 2, 3, 4, 5, 6, 7, 8] no axial force was applied; other researchers [2, 9, 10, 11, 12, 13, 14] induced an axial load, which was kept constant by force control during the lateral loading; the third group of researchers [15, 16] adopted a pseudo-constant vertical loading strategy, by applying an initial vertical load and, furthermore, restraining the rocking rotation of the wall element (displacement control) by a special device (pantograph). In the majority of the reported studies the vertical load level was related to the compressive strength (either design or characteristic/cylinder) of the concrete. The ratio of the applied axial load to the compressive capacity of the gross concrete section was referred to as [13] normalized axial load. A comparison of the applied axial load level reported by different authors is presented in Fig. 2 and Table 1, in terms of normalized axial stress, defined as the ratio of the induced compressive stress ($\sigma_0$) to the characteristic strength of concrete ($f_c$).

The seismic demand was imposed by in-plane quasi-static cyclic lateral loading, applied according to a predetermined pattern, which was referred to as horizontal loading history. A comparative study of the loading histories revealed the following characteristics: (1) all the investigators adopted symmetric cycle sequence, with increasing amplitude; (2) the loading cycles were either displacement controlled throughout the tests, or load
Table 1 Axial load levels reported in the literature

<table>
<thead>
<tr>
<th>Author</th>
<th>Induced stress ($\sigma_0$) [N/mm²]</th>
<th>Concrete strength mean values ($f_{ck}$) [N/mm²]</th>
<th>Normalized axial stress ($\sigma_0$/$f_{ck}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oesterle et al. (1984)</td>
<td>1.40</td>
<td>31.8</td>
<td>0.047</td>
</tr>
<tr>
<td></td>
<td>3.19</td>
<td>51.8</td>
<td>0.060</td>
</tr>
<tr>
<td></td>
<td>3.12</td>
<td>45.6</td>
<td>0.070</td>
</tr>
<tr>
<td></td>
<td>3.17</td>
<td>43.6</td>
<td>0.080</td>
</tr>
<tr>
<td></td>
<td>3.19</td>
<td>21.8</td>
<td>0.130</td>
</tr>
<tr>
<td>Lefas et al. (1990)</td>
<td>4.75</td>
<td>47.5</td>
<td>0.100</td>
</tr>
<tr>
<td>Satokoski et al. (1999)</td>
<td>7.86</td>
<td>39.3</td>
<td>0.200</td>
</tr>
<tr>
<td>Iso et al. (2000)</td>
<td>4.64</td>
<td>29.8</td>
<td>0.167</td>
</tr>
<tr>
<td>Palermo and Vecchio (2002)</td>
<td>6.06</td>
<td>27.8</td>
<td>0.220</td>
</tr>
<tr>
<td>Kizano et al. (2004)</td>
<td>3.59</td>
<td>21.3</td>
<td>0.167</td>
</tr>
<tr>
<td>Nagy-György et al. (2005)</td>
<td>0.50</td>
<td>30.0</td>
<td>0.017</td>
</tr>
<tr>
<td>Belmouden and Lestuzzi (2006)</td>
<td>2.30</td>
<td>67.5</td>
<td>0.034</td>
</tr>
</tbody>
</table>

Fig. 1 Experimental specimens

Table 2 Comparison of the reported axial load levels controlled up to the yield displacement ($\delta_y$), and thereafter displacement controlled; (3) the amplitude of a loading cycle in the displacement controlled range, was referred to as displacement level ($\delta_{j}$), storey drift angle ($R$), drift ratio ($R_{\delta}$), or displacement ductility level ($\mu_{\Delta}$); (4) the number of loading cycles on a displacement level ($n_j$) was reported to be 1, 2 or 3, but the same in a certain experiment; (5) the displacement increment between two consecutive displacement levels ($\Delta \delta_{j}$) was either constant or variable during the tests; and (6) the tests were stopped at the displacement level, referred to as deformation capacity ($\delta_u$), where the applied lateral load decreased by 10÷25 % of the recorded maximum value. In order to obtain commensurate loading histories, the loading cycle amplitudes were transformed in storey drift angles, according to the Eq.1-3:

\[ R = \frac{\delta_j}{h_w} [100 \times 10^{-3} \text{rad}] \]  

\[ \mu_{\Delta} = \frac{\delta_j}{\delta_y} \]  

\[ R_{\delta} = \frac{\delta_j}{h_w} [100 \%] \]

where $h_w$ is the wall height, however in certain cases it was measured from the level of the horizontal load. A representation of the studied loading histories in terms of half cycle envelope curves was given in Fig. 3.

The number of loading cycles in the elastic range ($n_0$) was recommended [17] to be large enough to obtain reliable data on the stiffness properties and it was suggested to be performed at two loading levels corresponding to 0.5$\delta_y$ and 0.75$\delta_y$.

The analysis of the presented loading histories exhibited significant scatter of the displacement increment ($\Delta \delta_{j}$) parameter. In the experimental studies [8, 14], the authors utilized a constant value for $\Delta \delta_{j}$, starting with the first displacement level ($\delta_j$), Eq. 4.

\[ \Delta \delta_{j} = \delta_{j+1} - \delta_j = \delta_j \]
In other tests, [6] and [10], the displacement increment was increased after certain cycles, according to Eq. 5 and Eq. 6.

\[
\Delta \delta_j = \delta_1; 2 \delta_1; 4 \delta_1
\]  
(5)

\[
\Delta \delta_j = 2 \delta_1; 3 \delta_1; 5 \delta_1
\]  
(6)

The yield displacement (\(\delta_y\)) of the specimens was associated with the displacement increment in [11], Eq. 7, which resulted in different displacement levels for each element.

\[
\Delta \delta_j = \delta_1 = \delta_y
\]  
(7)

The same procedure was designated in [2], except that the displacement increment was doubled after a certain number of cycles, Eq. 8, and the loading histories were identical for all elements.

\[
\Delta \delta_j = \delta_1 = \delta_y; 2 \delta_y
\]  
(8)

The cycle amplitudes were continuously increased in [15, 16], by the value of the previous displacement level, Eq. 9.

\[
\Delta \delta_j = \delta_j
\]  
(9)

Similar pattern was employed in [9], but after level “m” the increments were kept constant (Eq. 10).

\[
\Delta \delta_j = \delta_j; \delta_m
\]  
(10)

3 Vertical Load Evaluation

A five-storey Precast Reinforced Concrete Large Panel (PRCLP) building (Fig. 5) was analyzed to obtain data regarding the actual compressive stress level in the wall panels. The 3D geometric model of the building (Fig. 6) was sliced by horizontal planes at 5 cm intervals throughout the height. This resulted in a series of sections, which accounts for the variation of opening-solid relation in the wall panels along the vertical direction. By calculating the areas of cross sections, respectively the volumes and weights of 5 cm thick layers and subsequently summing the layer-weights starting from the top, it was possible to determine the weight of the bare structure above each section (Fig. 7). The computation was conducted separately on the internal and external walls, considering the supported slab areas (Fig. 5). Reinforced concrete specific weight of 24 kN/m\(^3\) was adopted as dead load (g) and 3 kN/m\(^2\) on slab was assumed as live load (q).

Henceforth the analysis was focused on the internal walls. Mean compressive stress along building height (Fig. 8) was figured out as the ratio of the above weight to the section area (only the actual area of concrete sections), taking into account the self weight of the above wall panels (g) and the total dead and live loads (g+q).
In this way $\sigma_0=0.89$ N/mm$^2$ mean compressive stress was obtained in internal walls at the ground floor, which corresponds to 0.056 mean normalized axial stress ($f_{ck}=16$ N/mm$^2$, according to the typical plan). The value of 0.056 was retained as reference, but it could not be employed for the detailed calculus, because it does not reproduce the two different vertical force transfer mechanisms in walls with openings, i.e. the axial force due to self weight of wall panels ($N_{0,gw}$), where index “gw” represents the dead load of structural walls, is directly proportional to sleeve wall cross section, exclusive of openings ($S_{w1}$), respectively the axial load received from the supported floor slabs ($N_{0,gs+qs}$), where index “gs” stands for self weight of slabs and “qs” stands for live loads on slabs, is proportional to the nominal wall cross section, inclusive of the opening area ($S_{w2}$). In Fig. 8 respectively Eq. 11 and 13 $\sigma_{0,gs+qs}$ was computed with the sleeve wall cross section ($S_{w1}$), and resulted in $\nu=0.056$ normalized axial stress, which don’t correlate to the actual force transfer mechanism, as stated above.

$$\nu = \nu_1 + \nu_2 = 0.021 + 0.035 = 0.056$$  \hspace{1cm} (11)

$$\nu_1 = \frac{\sigma_{0,gw}}{f_{ck}} = \frac{N_{0,gw}}{S_{w1} f_{ck}}$$  \hspace{1cm} (12)

$$\nu_2 = \frac{\sigma_{0,gs+qs}}{f_{ck}} = \frac{N_{0,gs+qs}}{S_{w1} f_{ck}}$$  \hspace{1cm} (13)

where $\nu$ is normalized axial stress, $\nu_1$ is the normalized axial stress due to wall panel self weight, $\nu_2$ is the normalized axial stress due to loads transferred from floor slabs.
### Table 2 Normalized axial stress assessment

<table>
<thead>
<tr>
<th></th>
<th>N0</th>
<th>N0,gw</th>
<th>N0,gs+qs</th>
<th>Sw1</th>
<th>Sw2</th>
<th>fck</th>
<th>σ0,gw</th>
<th>σ0,gs+qs</th>
<th>σ'0,gs+qs</th>
<th>σ0</th>
<th>ν1</th>
<th>ν2</th>
<th>ν'</th>
<th>ν''</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>8830.9</td>
<td>3270.3</td>
<td>5560.6</td>
<td>9.989</td>
<td>11.505</td>
<td>0.33</td>
<td>0.56</td>
<td>0.48</td>
<td>0.89</td>
<td>0.021</td>
<td>0.035</td>
<td>0.030</td>
<td>0.056</td>
<td>0.051</td>
</tr>
</tbody>
</table>

### Table 3 Presumptive axial loads

<table>
<thead>
<tr>
<th>Experimental element</th>
<th>fck [N/mm²]</th>
<th>S1 [cm²]</th>
<th>S2 [cm²]</th>
<th>S0 [cm²]</th>
<th>N1 [kN]</th>
<th>N2 [kN]</th>
<th>N [kN]</th>
<th>σ0 [N/mm²]</th>
<th>ν</th>
</tr>
</thead>
<tbody>
<tr>
<td>PRCWP 1, 2</td>
<td>18</td>
<td>3550</td>
<td>3550</td>
<td>3550</td>
<td>134</td>
<td>192</td>
<td>326</td>
<td>0.92</td>
<td>0.051</td>
</tr>
<tr>
<td>PRCWP 3, 4</td>
<td>18</td>
<td>3550</td>
<td>3550</td>
<td>2800</td>
<td>134</td>
<td>192</td>
<td>326</td>
<td>1.16</td>
<td>0.065</td>
</tr>
<tr>
<td>PRCWP 5, 6</td>
<td>18</td>
<td>3550</td>
<td>3550</td>
<td>1800</td>
<td>134</td>
<td>192</td>
<td>326</td>
<td>1.81</td>
<td>0.101</td>
</tr>
<tr>
<td>PRCWP 7</td>
<td>18</td>
<td>2800</td>
<td>3550</td>
<td>2800</td>
<td>106</td>
<td>192</td>
<td>298</td>
<td>1.06</td>
<td>0.059</td>
</tr>
<tr>
<td>PRCWP 8</td>
<td>18</td>
<td>1800</td>
<td>3550</td>
<td>1800</td>
<td>68</td>
<td>192</td>
<td>260</td>
<td>1.44</td>
<td>0.080</td>
</tr>
</tbody>
</table>

Note - \( f_{ck} \): characteristic strength of concrete; \( S_1 \): actual area of as-built wall cross section; \( S_2 \): nominal area of wall cross section; \( S_0 \): actual area of wall cross section; \( N_1 = \nu_1 f_{ck} S_1 \), axial load according to wall panel self weight; \( N_2 = \nu_2 f_{ck} S_2 \), axial load according to loads transferred from floor slabs; \( N = N_1 + N_2 \), eventual axial load on the experimental specimen; \( \sigma_0 = N / S_0 \), actual compressive stress in wall panels; \( \nu \): actual normalized axial stress.

Therefore, in Eq. 14 and 15 \( \sigma_{0,gs+qs} \) was substituted by \( \sigma_{0,gs+qs}' \) computed with nominal wall cross section \( (S_w2) \), which resulted in \( \nu' = 0.051 \) (Table 2).

\[
\nu' = \nu_1 + \nu_2' = 0.021 + 0.03 = 0.051 \quad (14)
\]

\[
\nu_2' = \frac{\sigma_{0,gs+qs}'}{f_{ck}} = \frac{N_{0,gs+qs}}{S_w2 f_{ck}} \quad (15)
\]

### 4 Axial Loading Strategy

The experimental test set-up (Fig. 9) was conceived to fulfill the requirements imposed by the pseudo-constant axial and in-plane reversed cyclic lateral loading strategy. To induce the vertical forces two hydraulic jacks were positioned on the upper (force transmitting) beam at 450 mm from the side edges of the experimental assembly. The hydraulic jacks were supported by vertical reaction frames, through steel rollers to enable the in-plane lateral (horizontal) displacements.

The proposed axial loading strategy includes the following steps:

- computation of the initial vertical load, which accounts for the normalized axial stress \( \nu' = 0.051 \) (invariant for all experimental elements);
- applying the initial vertical load by the two hydraulic jacks;
- restrain the rocking rotation of the experimental specimen during lateral forcing, by displacement control of the two vertical hydraulic jacks.

The applied initial vertical load depends on the characteristic strength of concrete \( (f_{ck}) \) of each wall specimen, on the day of experiment. For exemplification the initial axial loads were determined considering \( f_{ck} = 18 \text{ N/mm}^2 \) mean value (Table 3).

\[
\nu' = \nu_1 + \nu_2' = 0.021 + 0.03 = 0.051 \quad (14)
\]

\[
\nu_2' = \frac{\sigma_{0,gs+qs}'}{f_{ck}} = \frac{N_{0,gs+qs}}{S_w2 f_{ck}} \quad (15)
\]

### 5 Lateral Loading Strategy

The designation of the lateral loading history was based on the similar experimental works and recommendations presented in section x, taking into account the characteristics of the experimental program. Considering the novelty of the test set-up and the large number of the experimental variables, the simplest lateral loading history was assigned, to reduce the possibility of the misleading interpretations. Therefore, the decision was taken to utilize the same displacement controlled loading history for all experimental elements, irrespective of geometric configuration, reinforcement arrangement, and strengthening state. This resulted...
in the exclusion of the yield displacement (δy) based and dual controlled (load and displacement) loading histories. The displacement increment (Δδj) was proposed to be constant throughout the tests, according to Eq. 4.

In these conditions the loading history was determined by two parameters: the value of first displacement level (δ1) and the number of cycles on a displacement level (nj). In Table 4 the conversion factors between the displacement level (δj) and storey drift angle (R) were calculated, with wall height of hw=2150 mm.

Table 3 Displacement level – Storey drift angle conversion factors

<table>
<thead>
<tr>
<th>δj [mm]</th>
<th>R [x10⁻³ rad]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.465</td>
</tr>
<tr>
<td>2.15</td>
<td>1</td>
</tr>
</tbody>
</table>

Taking into account the constraints regarding the duration of the experimental test, e.g. a total cycle number of 20, multiplied by 15 min/cycle, results in 5 testing hours, the δ1=2.15 mm and nj=2 values were assigned. As failure criterion, the deformation capacity (δu) was associated with the displacement level, where 20% lateral load capacity drop is observed. The adopted loading history is presented in Fig. 10.

![Fig. 10 Proposed lateral loading history](image)

6 Conclusions

The pseudo-constant axial and quasi-static in-plane cyclic lateral loading strategy presented in this paper was conceived in accordance with the currently established testing methods, applied in several experimental research programs performed on reinforced concrete walls subjected to in-plane seismic loading conditions. The employed reference parameter, for the axial loads, was the normalized axial stress (ψ), i.e. the ratio of the induced stress (σi) to the characteristic strength of concrete (fck).

The comparative literature survey exhibited a data range of (0.01÷0.2) for ψ, while the gravity load analysis of a five-storey precast reinforced concrete large panel building indicated ψ=0.056 mean value for internal walls at ground floor. To account for the different vertical force transfer mechanisms in walls with openings, a relation was established, which assessed ψ to be 0.051, for the same building. The axial loading strategy proposed to be applied in the experimental tests corresponds to the pseudo-constant vertical loading method, i.e. initial axial loads are computed based on the adopted ψ'=0.051, these forces are induced in the walls by two vertical hydraulic jacks, and the rocking rotation of the experimental specimens during lateral forcing is restrained by the displacement control of the two jacks. Presumptive axial loads were regained from ψ', for the eight experimental elements, assuming the fck=18 N/mm² mean value. Concerning the lateral loads, several quasi-static cyclic loading histories were analyzed and compared in terms of the identified characteristic parameters. Although all related tests were performed in displacement control, the utilized loading histories presented significant scatter, revealed primarily by the displacement increment (Δδi) parameter. Being aware of the high number of experimental variables, in order to avoid misleading interpretations, the simplest lateral loading history was designated, identical for all experimental elements: two displacement controlled loading cycles on each displacement level (δi) with increments of Δδi=2.15 mm (corresponds to R=1x10⁻³ rad storey drift angle), constant throughout the test. The deformation capacity (δu) was associated with the displacement level, where 20% lateral load capacity drop is observed.

References:


End of the Special Session