NUMERICAL MODELING OF MULTI-STORY CONFINED MASONRY STRUCTURES IN SEISMIC REGIONS OF UKRAINE

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Abstract

In this paper some features of the numerical modeling of confined masonry buildings in final element software systems with taking into consideration the regional specifics of Ukraine’s seismic regions given.

Attention focused on difficulties encountered in the calculation of joint behavior of concrete frame elements with masonry due the requirements of the code "Construction in seismic regions of Ukraine", which caused by the specific technology. Guidance on the calculation and design of the confined masonry buildings elements provided.

Introduction

To date, the main part of the multi-storey buildings constructed in seismic regions of Ukraine is a reinforced concrete flat slab frame with diaphragms with the wall filling of lightweight concrete. On the second place confined masonry buildings (more than 25%). Despite a large amount of flaws that inherent to such buildings [1, 4-6]. Confined masonry buildings are in high demand especially for high-end housing or business class.

Problems of confined masonry buildings modeling relate to all it’s structural elements: vertical concrete and stone elements, horizontal floor elements. This article describes some issues related to the confined masonry buildings, which have been investigated over a fairly long period of time. For example, field studies of slabs with horizontal impact held in 1970-1972 [1].

Most part of data presented in this paper are the results of numerical analysis. All numerical studies described in the work carried out in software systems and LIRA and MONOMAH that implement the finite element method.

1. Modeling problems caused by overlaps behavior under lateral impacts

Today two types of ceiling panels in confined masonry buildings used: precast and monolithic. The most widely used (about 90-95% of cases) precast slabs because of their lower cost compared to monolithic, so the main focus of the work is given to the modeling of precast slabs.

Building code [2, p 3.3.1.], says: "slabs constructions should be made in the form of rigid horizontal discs securely connected to the vertical elements of the building and supporting their joint behavior under seismic loads." But the results of field and modeling studies conducted by Mikhailov AA [1] indicate that the design of this kind of slabs in their planes do not work as a rigid disc.

Building code [3] emphasizes that building’s malfunction may be caused by design method disadvantages, and the degree of reliability depends on the building’s calculation model.
Therefore, to determine the influence of slab’s stiffness in confined masonry building on its work under seismic impacts a number of numerical studies on computational models carried out.

The presented data describes an analysis of a ten floors building’s model (maximum number of floors established by code [2] is 10, buildings with the number of floors more than that fall under the category of experimental construction) with dimensions 12x60 m, walls grid 6x6 m, floor height 3m(Fig. 1-Fig. 2). At this stage of researches influences of base has not taken into account.

Fig. 1. Typical floor plan of building (created in the PC "MONOMAH")

Fig. 2. Общий вид расчетной схемы здания General view of the building’s design scheme

The calculation was performed on the seismic effects intensity 7 grades (corresponds to acceleration 0.1 m/s²) for the second category soils.

The following variations of overlap models were calculated:

1. Rigid slab with reduced modulus of elasticity and density(Fig. 3). Reduction was carried out by changing the modulus of elasticity, based on the bending work so that the solid concrete slab thickness of 22 cm was defined based on the equivalent stiffness and density cavity running from their plane (across the voids).
2. Overlap that consists of separate RC slabs, not related to each other.
3. Rigid orthotropic overlap with the reduced modulus of elasticity (similar to claim 1, but also takes into account the stiffness across the voids).
4. Rigid orthotropic overlap with the reduced modulus of elasticity. Reduction was carried out by changing the modulus of elasticity, based on the work plate bending so that the solid concrete slab thickness of 22 cm was defined based on the equivalent stiffness and density cavity operating in the plane (lengthwise and crosswise).
5. Orthotropic overlap with the reduced modulus of elasticity and a reduced value of the shear modulus (multiplied by 0.25 in accordance with [1]. Due to the possibility of poor work performance).

To simplify further discussion of the material all the schemes were assigned symbols (see Table 1).

![Precast slab](image)

**Fig. 3.** Precast slab

<table>
<thead>
<tr>
<th>№</th>
<th>Symbol</th>
<th>Scheme description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>RO</td>
<td>Rigid overlap with reduced modulus of elasticity and density</td>
</tr>
<tr>
<td>2</td>
<td>SO</td>
<td>Overlap of separate RC slabs, not connected with each other</td>
</tr>
<tr>
<td>3</td>
<td>ROO-1</td>
<td>Rigid orthotropic overlap with the reduced modulus of elasticity (stiffness also taken into account across the voids)</td>
</tr>
<tr>
<td>4</td>
<td>ROO-2</td>
<td>Rigid orthotropic overlap with the reduced modulus of elasticity, working in its plane (lengthwise and crosswise)</td>
</tr>
<tr>
<td>5</td>
<td>ROO-3</td>
<td>Rigid orthotropic overlap with the reduced modulus of elasticity and a reduced value of the shear modulus</td>
</tr>
</tbody>
</table>

Criteria according to which the results of calculation evaluated

1. Oscillation period.
2. Internal forces from seismic effects
3. Reinforcement of elements

The results of the internal forces in masonry and concrete columns analysis are not shown because the difference between them does not exceed 7%, due to the regularity of scheme. Therefore, changing the stiffness characteristics of the overlap
did not lead to a substantial redistribution of internal forces in the vertical bearing elements. Table 2 shows the results of the calculations.

### Table 2. Calculation results

<table>
<thead>
<tr>
<th>Parameter</th>
<th>RO</th>
<th>SO</th>
<th>ROO-1</th>
<th>ROO-2</th>
<th>ROO-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oscillation period, sek</td>
<td>1-st form</td>
<td>0.715</td>
<td>0.718</td>
<td>0.716</td>
<td>0.719</td>
</tr>
<tr>
<td></td>
<td>2-nd form</td>
<td>0.599</td>
<td>0.604</td>
<td>0.599</td>
<td>0.601</td>
</tr>
<tr>
<td></td>
<td>3-rd form</td>
<td>0.546</td>
<td>0.548</td>
<td>0.546</td>
<td>0.547</td>
</tr>
<tr>
<td></td>
<td>4-th form</td>
<td>0.221</td>
<td>0.252</td>
<td>0.221</td>
<td>0.222</td>
</tr>
<tr>
<td></td>
<td>5-th form</td>
<td>0.193</td>
<td>0.223</td>
<td>0.193</td>
<td>0.194</td>
</tr>
<tr>
<td>Maximum beam reinforcement(%)</td>
<td></td>
<td>0.511</td>
<td>0.681</td>
<td>0.601</td>
<td>0.621</td>
</tr>
<tr>
<td>Internal axial force in beam of top floor, (ton)</td>
<td>3.32</td>
<td>6.96</td>
<td>3.32</td>
<td>4.21</td>
<td>4.79</td>
</tr>
</tbody>
</table>

As a result of analysis of overlap model influence on such integral parameter of building as oscillation period we can conclude that it is minor. The greatest influence of the analyzed parameter has internal forces and reinforcement of beams. The maximum difference in internal axial forces was 110% (from 3.32 to 6.96 ton) for the scheme with tightly coupled teams plates (RO) compared to the scheme without connecting plates (SO). In comparison with scheme ROO-3 difference is 44%. The value for percent reinforcement varied smaller, respectively 33% and 37%. This discrepancy between the changes in internal efforts and reinforcement ratio is because an intensity of impact equals 7 and internal axial force in beams are small enough, and a major influence on their reinforcement provide vertical loads.

Similar results were obtained for other buildings configuration, but because of limited space, they are not shown here.

### 2. Modeling problems caused by reinforced concrete columns.

While foreign design experience suggests that vertical concrete elements must be positioned exactly at the intersection of walls, Ukrainian code [2] prescribes to locate concrete columns with an offset to make them open for viewing, at least on one side, and thus creates a considerable eccentricity. Difference in the approaches of codes can be visually assessed by Fig. 4 (on the lower figure (b) masonry not shown). On the scheme that made in accordance with DBN B.1.1-12:2006 visible eccentricities arising across a seismic building and in scheme configured in accordance with international standards, this effect is observed. Whereby in this paper we will carry out analysis of the effect of such position of a concrete reinforced columns in body of masonry. Evaluation criterias of this effect are such integrated dynamic performance of buildings (Table 1), the oscillations periods of the first three forms of natural vibrations and maximum displacements top of buildings, and internal forces (in particular shear force Qx).
Fig. 4. Location of reinforced concrete elements: 
a-b) according to the recommendations of the Research Institute of Earthquake Engineering in Oakland (CA) 
c) according to paragraph 3.10.14 Corresponding DBN 1.1-12:2006.

Four variants of 10-storey building models was calculated to check revealed discrepancies. Step of longitudinal and transverse walls 6m, size of the building 18 * 18 m, floor height of 3m. Material parameters: brick mark M75, mortar mark M50. These parameters are minimal masonry in accordance with [2] and is most often used in construction. 400x400 mm concrete columns of concrete C20/25. (Typical floor plan is shown in Fig. 5). Triangulation step finite element 0.2 x0.2 m

Fig. 5. Typical floor plan of the building, executed in PC MONOMAKH.

Also, taking into consideration that confined masonry buildings are built with two types of concrete slabs - monolithic or precast, in the numerical experiment were examined both types: monolithic slab (denoted to as "M" in Table 3) and precast (denoted by "P" in tabl3).
Table 3. Results of the calculations.

<table>
<thead>
<tr>
<th></th>
<th>M1</th>
<th>M2</th>
<th>P1</th>
<th>P2</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1, sek</td>
<td>1,129</td>
<td>1,146</td>
<td>1,203</td>
<td>1,206</td>
</tr>
<tr>
<td>T2, sek</td>
<td>1,129</td>
<td>1,132</td>
<td>1,069</td>
<td>1,086</td>
</tr>
<tr>
<td>T3, sek</td>
<td>1,034</td>
<td>1,033</td>
<td>0,935</td>
<td>0,9355</td>
</tr>
<tr>
<td>Ux, mm</td>
<td>26</td>
<td>28</td>
<td>25</td>
<td>26</td>
</tr>
<tr>
<td>Uy, mm</td>
<td>26</td>
<td>28</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Qx, tn/m</td>
<td>0,83</td>
<td>3,3</td>
<td>0,78</td>
<td>3,2</td>
</tr>
<tr>
<td>Qy, tn/m</td>
<td>-1,4</td>
<td>-2,2</td>
<td>0,84</td>
<td>2,7</td>
</tr>
</tbody>
</table>

However, due to the fact that the main purpose is to study the influence of the location of reinforced concrete columns in the body of masonry, in the parallel with different kinds of slabs scheme were considered to types of the location at the intersection of walls columns (denoted "1" in Table 3) and with the 400 mm displacement according to DBN (denoted by "2" in Table 3).

Analyzing the data presented in Table 3 it can be noted that the change in the dynamic characteristics of the main buildings is negligible (as in previous paragraph). However, there is a significant change in the transverse forces, which occurs because of the displacement of columns from the intersection of the walls on 400mm.

There are many suggestions on how this effect can also take into account in the calculation scheme. However, in our opinion it would be more productive to modify the design itself.

Summarizing global experience in designing of this kind of buildings and the disadvantages mentioned above in the location of reinforced concrete elements in the body of masonry, we can recommend the proposed location of the columns (Fig. 6-Fig. 8):

- to take column sections not less than 400x400 mm and not less than the cross section of the wall (at the intersection of the walls of the same thickness) or the smallest cross section of the wall (at the intersection of walls of different thickness);
- at L-intersection of walls to take column sections no less than 400x500 mm;
- Ensure quality control of concrete columns by increasing the cross-section, so for the survey was open at least 100 mm.

![Diagram](image)

Fig. 6. Proposed location of reinforced concrete columns in the body of wall at the T-intersection
3. **Modeling problems caused by masonry**

One of the most difficult issues in the modeling of confined masonry structures is masonry with openings. This is due to the fact that at the corners of openings emerge significant tensile stresses which considerably exceed limited ones (Fig. 9).
There are several approaches for solving this problem:

a. accounting in the calculation scheme window lintel (Fig. 10);

b. accounting in the calculation scheme interwindow pillar (masonry parts above and under the window accounted as loads) (Fig. 11);

c. stepwise calculation, with the exception from the design scheme of finite elements, which stresses exceed the limit values (simplified nonlinear analysis).

Each of these approaches has a number of positive and negative sides. For example, in order to simulate real walls behavior the presence of lintels can be taken into account. It leads (variant a.) to a local redistribution of internal forces in masonry at the upper window corners, wherein the stresses of the sill masonry part in remains unchanged (Fig. 10).

Fig. 9. Stresses in the masonry from the seismic load

Fig. 10. Stresses in the masonry from the seismic load (scheme with lintels)
b. **Design scheme**, which takes into account only the interwindow pillars and above the window, sill and part simulated by load, is more flexible than the original scheme. However, this scheme takes into account one of the worst-case scenarios: as a result of seismic effects cracks emerge in the corners of the openings and this part is removed from the work of whole masonry (Fig. 11).

![Stresses in the masonry from the seismic load](image)

**Fig. 11.** Stresses in the masonry from the seismic load (scheme without sills and upper windows parts)

**c. Stepwise calculation**, with the exception from the design scheme of finite elements, efforts which exceed the limit values (Simplified nonlinear analysis) One of the main difficulties in Ukraine used software systems is the inability to perform nonlinear dynamical calculations so in some cases it is necessary to simulate nonlinear analysis with a series of successive linear calculations with the exception of a finite element stress that exceeds the limit. Such a calculation, ultimately leads to a result very close to the one considered in paragraph b.

Therefore, the calculation of confined masonry buildings to determine the internal forces in the masonry it is appropriate to the scheme in which the work is modeled walls only interwindow pillars.

**Conclusions:**

The paper considers the main difficulties that are familiar for engineers that calculate and design confined masonry buildings in seismic regions.

2. Variants of prefabricated slabs taking into consideration described

3. Proposed solution for eccentric arranged concrete columns in the body of masonry elements.

4. The main difficulties that arise in the calculation of masonry considered. The ways of solving that difficulties suggested.
References


