Studying Seismic Stability of Buildings Constructed Using Lift-Slab Method and Equipped with Rubber-Steel Seismic Isolators

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Studying Seismic Stability of Buildings Constructed Using Lift-Slab Method and Equipped with Rubber-Steel Seismic Isolators

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Abstract. As the construction industry develops, unique unconventional methods of improving the seismic stability of buildings are used. One of these methods is the seismic isolation of buildings using special isolating supports. Rubber steel supports with high damping ability can improve the seismic stability of the building by 1.5 times on average. Since the seismic stability of the buildings constructed using the lift-slab method was set at up to 7 points, we expect to increase it to 8-9 points through the installation of the rubber steel supports in these buildings. The implementation of the current seismic resistance improvement methods for the existing buildings and structures shall help achieve the following: improve the preparation of the authorities and residents to major earthquakes; significantly reduce seismic risks and the consequences of both major earthquakes and industrial disasters caused by earthquakes; provide the economic and social stability of the country in case of emergencies.

1. Introduction
One of the ways to improve the modern construction industry is combining cast-in-place construction and prefabricated concrete elements. This will help select and use the advantages of each of these construction technologies, which will, in turn, result in the following:
- the reduction and optimization of construction material consumption;
- the increase in the quality of structures due to their factory fabrication;
- the increase in the construction rates due to the use of prefabricated structures and the reduction of formworks, reinforcing, and concrete operations during construction.

This is manifested in the application of the lift-slab method, which is a promising development vector in industrialized construction.

1.1. Architectural and engineering design of the building in question
In 1975, the authors designed a Krest type 16-storey 120-apartment residential building with the possibility of using it as a standalone or linked unit. This building was designed for Climate Region 4
with a seismic rating of 7 points. The building is shaped as a cross with four equally developed wings. The building is perceived as a set of standalone vertical structures because the wigs are shaped like petals, which gives make the building look light and neat. The visual impact of the building is strengthened by the combination of flat and blind outer walls in the vertical cut faced with natural stone and the projected sunrooms with curved concrete railings. The building has a unique and attractive shape.

The structural volume of the building is 39058 m$^3$, its equalized total floor area is 8119 m$^3$, its living floor area is 4424 m$^2$, and the coverage is 805 m$^2$ [5].

The centered design helped produce a compact building with eight two- or three-room flats per one vertical transportation core. The set of apartments can be changed depending on the specific requirements. The available options range from one-room to four-room apartments. The key geometry parameters and wing layouts allow for the linking of such buildings with each other and other structures.

Staircases, elevators, and other vertical communications of the building are located in the stiffening core. Elevators and staircases are accessible from a common space connected to two corridors providing access to four apartments each. All of the apartments have natural aeration, windows on two sides, and their sun exposure complies with the applicable standards, which creates comforts typical of multi-bay houses. In the apartments, the authors used the daytime and nighttime room zoning principle. The living room and the kitchen are located near the entrance and have a shared sunroom. Bedrooms with their sunrooms are grouped around the bathroom. The hall is an extra shared area for the four apartments that have access to it.

The spatial design features air zones on every floor. Due to them, the staircase and elevator sections receive natural light through the openings in the walls of the stiffening core. They are kept smoke-free due to the smoke removal through the air zones and the air overpressure in the halls. The floor slabs in the air zones feature hatchways with a ladder to evacuate people from top storeys to the second floor.

The first floor is designed to house shops and various amenity rooms. Besides, utility rooms like the refuse chute termination room, switchboard room, and the control unit are located here.

The support structure of the building is a reinforced concrete framework made using the shear wall system. Its spatial stiffness is ensured by the reinforced concrete stiffening core in the center (Figure 3.1).

![Figure 1. Structural scheme of a 16-storey 120-apartment residential building of the Krest type 1-column frame; 2-floor slab; 3-stiffening core; 4-barrel column.](image-url)
The reinforced concrete framework of the building consists of 36 built-up columns, four of which are located inside the stiffening core, and flat floor slabs shaped as a symmetrical cross covering the entire floor area with an octagonal aperture in the center for the stiffening core. The area of one floor slab is 697 m$^2$.

The spatial stiffness of the building is achieved by the joint work of the reinforced concrete core and the framework. These are pin-jointed through the cantilever projections of all floor slabs. The reinforced concrete stiffening core of the Krest type building is a thin-walled hollow structure with apertures. The outer perimeter of the core is a regular octagon obtained by the parallel displacement of the outer perimeter to the wall thickness, taking into account their swelling at apertures. The apertures are located on two opposite faces of the core at regular vertical intervals on all of the building floors.

Dimensions, wall thickness, and other stiffness core parameters are calculated to ensure the required building robustness and stiffness taking into account that it will be fitted with staircases and elevators with halls and other vertical communications.

The stiffening core with four columns located inside and eight columns located outside of it has a common footing shaped like a round ribbed slab made of cast-in-place reinforced concrete. Column footings of standalone bases and the stiffening core base are connected at the top with framing beams that facilitate the joint work of all the base elements under horizontal loads. The framing beams located at the perimeter act as foundations for the outer wall of the first floor. Grade M200 heavy concrete was used for the base works.

The building design in question features an independent framework within the core, which allows for the erection of the core with internal floor slabs irrespective of the structures outside of the stiffening core. This design allows for a significant reduction of construction times because it allows for both the simultaneous construction of the core and the framework and the construction of the core before the rest of the building.

The framework inside the stiffening core consists of four columns and floor slabs. Columns have a cross-section of 45x45 cm on all levels. The columns of the first level are 12.6 m long and made of M400 heavy concrete, while other columns are 2.93 m long and made of M300 concrete. The first-level columns have different lengths and aperture locations for the installation of elevators, crane platforms, and floor slabs inside the core at intermediary and design locations.

The floor slabs are flat, 18 cm thick, and made of M200 concrete. Their shape is determined by the internal configuration of the core and the locations of vertical communications [5].

The staircases and their intermediate landings are made of prefabricated reinforced concrete panels. They are supported by floor slabs at one end and reinforced concrete beams at the other. The beams, in their turn, are supported by the staircase walls made of prefabricated reinforced concrete panels. The walls of the elevator shaft are also made of prefabricated panels with two cab access apertures on all floors. The staircase and elevator group structure and linked to the columns and floor slabs inside the core. Staircases and elevators are accessed from the same hall on all floors. These halls are adjoined by corridors with constant air overpressure to facilitate the smoke removal from the staircase and elevator group. The corridors are separated from the halls with walls and self-closing glass doors. The same doors are used in the core apertures connecting the corridors and the air zones. Garbage chute hatches are located on the intermediate landings. Floor slabs feature apertures for the air ducts creating overpressure in corridors. They also have apertures for rainwater pipes. Rainwater pipe channels are also implemented in the core. The core also features channels for other vertical communications.

The Krest type 16-storey residential building is constructed in increments. The construction of the stiffening core is a bit faster than floor slab lifting. Connection structures and elements are the same as for the Trilistnik type 16-storey residential building.
2. Materials and methods

2.1. The theoretical justification of damper calculations

![Diagrams of SRMOS types](image)

Figure 2. Types of SRMOS: a - flanged; b – unflanged.

Depending on the damping parameters, there can be SRMOS with low (Figure 2) and high damping (Figure 3).

![Hysteresis loops for SRMOS](image)

Figure 3. SRMOS hysteresis loops: a - for supports with low damping. 
  b - for supports with high damping

Flanged SRMOS can withstand multiple stress, strain, displacement, and torsion load cycles.

The SRMOS produced and used today in the Republic of Armenia [3;4] are unflanged. Their geometry, as well as physical and mechanical parameters, are shown in Tables 1 and 2. The overall view of support attachment to building structures is shown in Figure 3.4.
Figure 4. SRMOS currently used in Armenia [2];
a - geometric dimensions; b - attachment point of the support to the building structures

The seismic isolator is attached to the structure through support rings that only hold it horizontally, i.e. the support itself is lying freely between the building structures.

Table 1. Geometric characteristics of SRMOS used in Armenia.

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Units</th>
<th>Notation</th>
<th>Value</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Support outer diameter</td>
<td>mm</td>
<td>D</td>
<td>380</td>
<td>± 2.0</td>
</tr>
<tr>
<td>2</td>
<td>Inner diameter of the support</td>
<td>mm</td>
<td>d₁</td>
<td>19</td>
<td>± 1.0</td>
</tr>
<tr>
<td>3</td>
<td>Support height</td>
<td>mm</td>
<td>H</td>
<td>202.5</td>
<td>± 2.5</td>
</tr>
<tr>
<td>4</td>
<td>Thickness of rubber layers</td>
<td>mm</td>
<td>S</td>
<td>9</td>
<td>± 0.1</td>
</tr>
<tr>
<td>5</td>
<td>Thickness of steel sheets</td>
<td>mm</td>
<td>S₁</td>
<td>2.5</td>
<td>± 0.1</td>
</tr>
<tr>
<td>6</td>
<td>Thickness of steel sheets</td>
<td>mm</td>
<td>S₂</td>
<td>360</td>
<td>± 0.5</td>
</tr>
<tr>
<td>7</td>
<td>Support ring diameter</td>
<td>mm</td>
<td>d₃</td>
<td>376</td>
<td>± 0.5</td>
</tr>
<tr>
<td>8</td>
<td>Support ring thickness</td>
<td>mm</td>
<td>S₃</td>
<td>20</td>
<td>± 0.2</td>
</tr>
<tr>
<td>9</td>
<td>Surface protective layer</td>
<td>mm</td>
<td>S₃’</td>
<td>2</td>
<td>± 0.1</td>
</tr>
<tr>
<td>10</td>
<td>Support weight</td>
<td>kg</td>
<td>–</td>
<td>77.5</td>
<td>± 2.5</td>
</tr>
</tbody>
</table>

Table 2. Physical and mechanical characteristics of SRMOS used in Armenia.

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Units</th>
<th>Value</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Rubber shear modulus</td>
<td>MPa</td>
<td>0.97</td>
<td>± 0.15</td>
</tr>
<tr>
<td>2</td>
<td>Support vertical stiffness</td>
<td>kN/mm</td>
<td>300</td>
<td>at least</td>
</tr>
<tr>
<td>3</td>
<td>Horizontal stiffness of the support</td>
<td>kN/mm</td>
<td>0.81</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>Horizontal calculated displacement</td>
<td>mm</td>
<td>280</td>
<td>Up to</td>
</tr>
<tr>
<td>5</td>
<td>Vertical design support load</td>
<td>kN</td>
<td>1500</td>
<td>Up to</td>
</tr>
</tbody>
</table>
This type of seismic isolator can withstand multiple stress and displacement load cycles. When the support takes the load from the building weight, its vertical movements do not normally exceed several millimeters. However, if horizontal vibrations occur due to seismic impacts, displacement deformations may reach several dozens of centimeters.

According to [4], design diagrams of buildings and structures normally look like fully-fixed weightless rods holding concentrated masses and fluctuating along one of the key symmetry axes (Figure 5). Seismic forces are applied to structures statically, and the dynamic properties of buildings are considered using the dynamic response factor.

![Design diagram of the building](image)

**Figure 5.** Design diagram of the building.

According to [3], the calculated value of the horizontal seismic load $S_{ki}$ applied to point $k$ and complying with form $i$ of free motion of the building or structure can be determined using the following formula:

$$S_{ki} = k_1 k_2 k_3 S_{oki},$$  \hspace{1cm} (1)

where $S_{oki}$ is the horizontal seismic load for form $i$ of structure oscillations determined based on the elastic deformation assumption using the following formula:

$$S_{oki} = Q_k A k_1 k_2 k_3 \eta \beta,$$  \hspace{1cm} (2)

where: $Q_k$ is the load causing the inertia force focused on point $k$;
$A$ is a non-dimensional seismic rating reflecting the ratio between the calculated acceleration of the ground in the settlement in question and the acceleration of gravity;
$k_1$ is the coefficient accounting for the permissible building and structure damages;
$k_2$ is the importance coefficient of the building and structure;
$k_3$ is the interaction coefficient for the base and the structure;
$k_0$ is the non-dimensional ground condition coefficient;

$\beta_i$ is a non-dimensional dynamic response factor complying with form $i$ of free oscillations of the building or structure in question (Figure 6);

$\eta_{ki}$ is a non-dimensional coefficient depending on the ordinates of the free oscillations form $X_{ki}$ and the concentrated load values $Q_k$ (aka the oscillation form coefficient).

\begin{align*}
\beta_{i} & = k_{0} \eta_{ki} \beta_i \left( T_i / 2 \pi \right)^2, \\
\Delta_{ki} & = 0.8 \left( x_{(k+1)i} - x_{ki} \right).
\end{align*}

The maximum values of the horizontal motions of floor $k$ for form $i$ of oscillations $x_{ki}$ and floor deflection $\Delta_{ki}$ are determined using the following formulae:

\begin{align}
\frac{\eta_{ki}}{2 \pi} k_{0} & = \frac{Q}{\pi K_{\text{eff}} g}, \\
\beta_{i} & = \frac{\Delta_{ki}}{x_{ki}}.
\end{align}

The values of the free oscillations period $T$ for buildings and structures featuring seismic isolation systems with horizontal stiffness and the respective efficient seismic isolator stiffness are calculated using the following formula:

\begin{equation}
T = 2 \pi \sqrt{\frac{Q}{K_{\text{eff}} g}},
\end{equation}

where $Q$ is the total vertical static load (superstructure weight) taking into account the combination factors determined according to [1];

$K_{\text{eff}}$ is the stiffness of the seismic isolation system that equals the sum of efficient stiffnesses of all the seismic isolators in the system determined using the specifications provided by the manufacturers;

$g$ is the acceleration of gravity.
2.2. Calculations

![Figure 7. Calculation model of an existing building.](image)

1. First of all, we had to determine the weight of the building. We used it to calculate the required damper weight. In this case, it was the weight of the top floor panel resting on rubber steel supports. According to the seismic standards of the Republic of Armeria, SNRA II-6.02-2006 Antiseismic Construction. Design Standard, Yerevan, 2006 - 62 p, the weight of the damper has to be at 7% of the building weight.

2. We calculated the oscillations period for a building without a damper.

3. We used the standards for the horizontal load per one cushion to calculate the number of cushions.

4. We calculated the displacement of the building with and without the damper.

5. We compared and calculated the improvement of the seismic stability of the building with a flexible top floor.
Figure 8. design model of the existing building with the flexible top floor.
The calculations for the Krest type 16-storey building with and without damper were carried out using the spectral theory in the LIRA SAPR software package. The oscillation damper consists of 80 rubber steel supports that hold a monolithic reinforced concrete slab of 80 cm thick. According to the theory, the weight of the oscillation damper has to be about 7% of the building weight, and its oscillation period must equal the Form I building oscillation period. For a regular building, oscillation periods are 0.92 seconds for Form I (Figure 10) and 0.25 for Form II. For the building with a damper, there are two basic oscillation forms. However, in this case, the first basic form is divided into two subforms with oscillation periods of 1.13 and 0.7 seconds. If the oscillation period for Form I is 1.13 seconds, the damper and the building move in the same direction (Figure 11). If the oscillation period for Form I is 0.7 seconds, the damper and the building move in the opposite directions (Figure 12).
Thus, earthquake energy damping mainly takes place in the rubber steel supports, which reduces the forces applied to the building framework. As for the second basic oscillation form, it practically didn't change (0.25 became 0.246), which means that the damper parameters, namely the ratio of the damper weight and the building weight and the number of rubber steel supports, were selected quite accurately.
To assess the difference between the forces applied to the building due to a seismic impact before and after the installation of the damper, review the values of total transversal forces at the building base level. In the regular building, this parameter is 7,554 t.s., and for the building with the damper featuring two Form I subtypes, it is 5,246 and 2,922 t.s. respectively. Thus, the formation of two Form I subtypes resulted in a significant reduction of total transversal forces at the building base level.

The charts below show the differences in the forces in specific building elements.

Oscillation periods in the building without the damper calculated in the LIRA SAPR software package

**BUILDING WITHOUT DAMPER**

**PROPER VALUES, FREQUENCIES, OSCILLATION PERIODS, LOADINGS 2 (model 32)**

<table>
<thead>
<tr>
<th>No.</th>
<th>PROPER</th>
<th>FREQUENCIES</th>
<th>PERIODS</th>
<th>COEFFICIENT OF DYNAMIC</th>
<th>MODAL DISTRIBUTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RAD/S</td>
<td>HZ</td>
<td>S</td>
<td>%</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.148256</td>
<td>6.75</td>
<td>1.07</td>
<td>0.9310</td>
<td>3.184808 0.6</td>
</tr>
<tr>
<td>2</td>
<td>0.146681</td>
<td>6.82</td>
<td>1.09</td>
<td>0.9212</td>
<td>-32.960004 65.6</td>
</tr>
<tr>
<td>3</td>
<td>0.042610</td>
<td>23.47</td>
<td>3.74</td>
<td>0.2676</td>
<td>0.094563 66.2</td>
</tr>
<tr>
<td>4</td>
<td>0.039804</td>
<td>25.12</td>
<td>4.00</td>
<td>0.2500</td>
<td>16.370979 82.4</td>
</tr>
<tr>
<td>5</td>
<td>0.036967</td>
<td>27.05</td>
<td>4.31</td>
<td>0.2322</td>
<td>1.276591 82.5</td>
</tr>
<tr>
<td>6</td>
<td>0.035174</td>
<td>28.43</td>
<td>4.53</td>
<td>0.2209</td>
<td>-0.000259 82.5</td>
</tr>
<tr>
<td>7</td>
<td>0.034428</td>
<td>29.05</td>
<td>4.63</td>
<td>0.2162</td>
<td>-0.198347 82.5</td>
</tr>
<tr>
<td>8</td>
<td>0.033709</td>
<td>29.67</td>
<td>4.72</td>
<td>0.2117</td>
<td>-1.238446 82.6</td>
</tr>
<tr>
<td>9</td>
<td>0.032848</td>
<td>30.44</td>
<td>4.85</td>
<td>0.2063</td>
<td>0.036065 82.6</td>
</tr>
<tr>
<td>10</td>
<td>0.032586</td>
<td>30.69</td>
<td>4.89</td>
<td>0.2046</td>
<td>0.020430 82.6</td>
</tr>
</tbody>
</table>

Oscillation periods in the building with the damper calculated in the LIRA SAPR software package

**BUILDING WITH DAMPER**

**PROPER VALUES, FREQUENCIES, OSCILLATION PERIODS, LOADINGS 2 (model 32)**

<table>
<thead>
<tr>
<th>No.</th>
<th>PROPER</th>
<th>FREQUENCIES</th>
<th>PERIODS</th>
<th>COEFFICIENT OF DYNAMIC</th>
<th>MODAL DISTRIBUTION</th>
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</thead>
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<td></td>
<td>RAD/S</td>
<td>HZ</td>
<td>S</td>
<td>%</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.182542</td>
<td>5.48</td>
<td>0.87</td>
<td>1.1464</td>
<td>2.529230 0.4</td>
</tr>
<tr>
<td>2</td>
<td>0.180420</td>
<td>5.54</td>
<td>0.88</td>
<td>1.1330</td>
<td>29.836667 50.3</td>
</tr>
<tr>
<td>3</td>
<td>0.111660</td>
<td>8.96</td>
<td>1.43</td>
<td>0.7012</td>
<td>-18.381654 19.0</td>
</tr>
<tr>
<td>4</td>
<td>0.110998</td>
<td>9.01</td>
<td>1.43</td>
<td>0.6971</td>
<td>0.342168 69.3</td>
</tr>
<tr>
<td>5</td>
<td>0.042771</td>
<td>23.38</td>
<td>3.72</td>
<td>0.2686</td>
<td>0.011588 69.3</td>
</tr>
<tr>
<td>6</td>
<td>0.039765</td>
<td>25.15</td>
<td>4.00</td>
<td>0.2497</td>
<td>0.055708 69.3</td>
</tr>
<tr>
<td>7</td>
<td>0.039214</td>
<td>25.50</td>
<td>4.06</td>
<td>0.2463</td>
<td>16.051238 83.8</td>
</tr>
<tr>
<td>8</td>
<td>0.036324</td>
<td>27.53</td>
<td>4.38</td>
<td>0.2281</td>
<td>-1.386731 83.9</td>
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<td>9</td>
<td>0.035057</td>
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<td>4.54</td>
<td>0.2202</td>
<td>0.025224 83.9</td>
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<td>10</td>
<td>0.032176</td>
<td>31.08</td>
<td>4.95</td>
<td>0.2021</td>
<td>-0.603043 83.9</td>
</tr>
</tbody>
</table>

Loading 1 – Vertical static loads
Loading 2 – Seismic impact along the X-axis
Loading 3 – Seismic impact along the Y-axis
BUILDING WITHOUT DAMPER

Total joint loads on the main framework:
Loading 1 PX=1.6591e-021 PY=0 PZ=19983.9 PUX=0.0183846 PUY=0.0030985 PUZ=0
Loading 2-2 PX=-7554.29 PY=738.617 PZ=-5.52449 PUX=0 PUY=0 PUZ=0
Loading 2-4 PX=-2629.31 PY=226.294 PZ=12.7085 PUX=0 PUY=0 PUZ=0
Loading 3-1 PX=-711.634 PY=-7241.68 PZ=-0.610838 PUX=0 PUY=0 PUZ=0
Loading 3-5 PX=-202.603 PY=-2576.41 PZ=25.2871 PUX=0 PUY=0 PUZ=0
Loading 3-7 PX=-11.3141 PY=-331.661 PZ=21.6074 PUX=0 PUY=0 PUZ=0

BUILDING WITH DAMPER

Total joint loads on the main framework:
Loading 1 PX=6.62736e-021 PY=-2.37528e-017 PZ=21439.8 PUX=-0.0190498 PUY=0.00470096 PUZ=1.21871e-008
Loading 2-2 PX=-5245.54 PY=445.429 PZ=-3.01856 PUX=0 PUY=0 PUZ=0
Loading 2-3 PX=-2922.59 PY=70.6868 PZ=-2.73734 PUX=0 PUY=0 PUZ=0
Loading 2-7 PX=-2527.61 PY=240.692 PZ=13.0895 PUX=0 PUY=0 PUZ=0
Loading 3-1 PX=-436.582 PY=-5104.22 PZ=-0.259478 PUX=0 PUY=0 PUZ=0
Loading 3-4 PX=-52.9487 PY=2755.27 PZ=-0.0630642 PUX=0 PUY=0 PUZ=0
Loading 3-8 PX=-226.872 PY=-2728.26 PZ=4.83185 PUX=0 PUY=

2.2. Calculations

The weight of the damper has to be approximately ≈7% of the building weight.

\[ Q_2 = 7\% \cdot Q_{bd} = 0.007 Q_{bd} \]

\[ Q_{bd} = 19984 \text{ t.} \Rightarrow Q_2 = 0.07 \cdot 19984 \text{ t.} = 1398.88 \text{ t.} \approx 1400 \text{ t.} \]

The damper oscillation period must be the same as the building oscillation period. (Form I)

\[ T_2 = T_{bd} I ; T_{bd} I = 0.92 \text{ sec} \]

\[ T_2 = T_{bd} = 2\pi \sqrt{\frac{Q_2}{9 \cdot C_2}} \Rightarrow C_2 = \frac{4 \pi^2 \cdot Q_2}{T_2^2 \cdot g} = \frac{4 \pi^2 \cdot 1400}{0.92^2 \cdot 9.81} = 6650 \text{ t/m} \]

The horizontal stiffness of one SRMOS (used in Armenia) is 81 t/m.

Thus, the number of SRMOS is as follows:

\[ n = \frac{6650}{81} = 82 \text{ pcs.} \]

Taking into account the design solutions (support dimensions, their locations, the number of columns and walls), the number of SRMOS is 80 pcs.

Thus, the damper oscillation period is as follows:

\[ T_2 = 2\pi \sqrt{\frac{Q_2}{9 \cdot C_2}} = 2\pi \sqrt{\frac{1400}{9.81 \cdot 81 \cdot 80}} = 0.93 \text{ sec.} \]

We used a monolithic reinforced concrete slab as a damper. Its thickness is as follows:

\[ h = \frac{Q}{A_{ar.} \cdot R_6} = \frac{1400}{714 \cdot 2.5} = 0.78 \text{m} = 78 \text{ cm} \]

we adopt 80 cm.

Results: damper weight – 1400 t. (Figure 13), the damper is an 80-cm thick reinforced concrete slab. The number of SRMOS is 80 pcs (with the stiffness of 6650 t/m)
Figure 13. 80-cm thick reinforced concrete plate damper.

According to the seismic standards of the Republic of Armenia, rubber steel cushions can only be located on the carrying structures (columns, diaphragms, and carrying walls). Figure 14 shows a layout of a typical floor of a Krest type building constructed using the lift-slab method with columns and carrying walls. We used this layout to determine the locations of the rubber steel support cushions. Since the carrying structures of the Krest type buildings are columns and carrying walls, we will place the rubber steel cushions on them.
According to the calculations, the required number of SRMOS is 82 pcs. Taking into account the design solutions (support dimensions, their locations, the number of columns and walls), the number of SRMOS is 80 pcs (Figures 15, 16).

Thus, the damper oscillation period is as follows:

$$T_2 = 2\pi \sqrt{\frac{Q_2}{9 \cdot C_2}} = 2\pi \sqrt{\frac{1400}{9.81 \cdot 81 \cdot 80}} = 0.93 \text{ sec.}$$
According to the results of the calculations performed in the Lira Sapr software, the maximum framework displacement due to seismic impacts for the building with a damper is 226 mm. To avoid the collision of the damper and the stiffening core, we leave a gap of 300 mm between the damper and the building framework.

3. **Total transversal force at the building base level**

Building without damper

2-2. \( Q_{1c,a} = 7554 \) t. (Form I)

2-4. \( Q_{2c,a} = 2629 \) t. (Form II)

Building with damper

2-2. \( Q_{11a} = 5246 \) t. (Form I)
2.4. $Q_{1,d} = 2922$ t. (Form I)
2.7. $Q_{2,d} = 2527$ t. (Form II)

1. The transversal forces at the base level for the building without the damper (Form I) and with the damper (Form I-I) reduced:
   $\Delta Q_{1,1} = Q_{1c,d} - Q_{1-d} = 7554 - 5246 = 2308$ t.
2. The transversal forces at the base level for the building without the damper (Form I) and with the damper (Form I-II) reduced:
   $\Delta Q_{1,2} = Q_{1c,d} - Q_{1-2d} = 7554 - 2922 = 4632$ t.
1. The transversal forces at the base level for the building without the damper (Form I) and with the damper (Form II) reduced:
   $\Delta Q_2 = Q_{2c,d} - Q_{2d} = 2629 - 2527 = 102$ t.

Oscillation periods
- Building without damper
2.2 – 0.92 sec. (Form I)
2.4 – 0.25 sec. (Form II)
Building with damper
2.2 – 1.13 sec. (Form I)
2.3 – 0.7 sec. (Form I)
2.7 – 0.246 sec. (Form II).

MOMENT DIAGRAMS FOR A TYPICAL FLOOR

Diagram of a My building without damper (t*m)

Diagram of a My building with a Form 1 damper (t*m)
3.1. Building displacement patterns

X-axis displacement patterns for a building without a damper (mm)

X-axis displacement patterns for a building with a Form 1-1 damper (mm)
3.2. Stress patterns

Stress patterns for a My building without a damper (t*m)
Stress patterns for a My building with a Form 1-1 damper (t*m)

Stress patterns for a My building with a Form 1-2 damper (t*m)
3.3. SRMOS displacement patterns

Patterns of SRMOS displacements along the X-axis, Form 1-1 (mm)

Patterns of SRMOS displacements along the X-axis, Form 1-2 (mm)

4. Results
The study of the seismic stability of the buildings constructed using the lift-slab method and equipped with rubber-steel seismic isolators produced the following results:

1. For the building without the damper, oscillation periods are 0.92 seconds for Form 1 and 0.25 seconds for Form 2.
2. For the building with the damper, the oscillation period is 1.13 seconds for the first subtype when the damper and the building move in the same direction.
3. For the building with the damper, the oscillation period is 0.7 seconds for the second subtype when the damper and the building move in opposite directions.
   Thus, earthquake energy damping mainly takes place in the rubber steel supports, which reduces the forces applied to the building framework.

5. **Conclusions**
The implementation of the current seismic resistance improvement methods for the existing buildings and structures shall help achieve the following:
   1. improve the preparation of the authorities and residents for major earthquakes;
   2. significantly reduce seismic risks and the consequences of both major earthquakes and industrial disasters caused by earthquakes;
   3. provide the economic and social stability of the country in case of emergencies.

6. **Conclusions and discussion**
The results of the study of the seismic stability of the buildings constructed using the lift-slab method and equipped with rubber-steel seismic isolators made us conclude the following:
   4. For the building without the damper, oscillation periods are 0.92 seconds for Form 1 and 0.25 seconds for Form 2.
   5. For the building with the damper, the oscillation period is 1.13 seconds for the first subtype when the damper and the building move in the same direction.
   6. For the building with the damper, the oscillation period is 0.7 seconds for the second subtype when the damper and the building move in opposite directions.
      Thus, earthquake energy damping mainly takes place in the rubber steel supports, which reduces the forces applied to the building framework.
      As for the second basic oscillation form, it practically didn't change (0.25 became 0.246), which means that the damper parameters, namely the ratio of the damper weight and the building weight and the number of rubber steel supports, were selected quite accurately.
      To assess the difference between the forces applied to the building due to a seismic impact before and after the installation of the damper, review the values of total transversal forces at the building base level.
      1. In the building without the damper, this parameter is 7,554 t.s., and for the building with the damper featuring two Form I subtypes, it is 5,246 and 2,922 t.s. respectively. Thus, the formation of two Form I subtypes resulted in a significant reduction of total transversal forces at the building base level.
      The results of the master's degree thesis research contain the data that show that it is possible to use flexible top floor technology as one of the ways to solve the problem of reducing seismic risks in Yerevan. This problem is recognized by the Government of the Republic of Armenia by order No. 392 of June 7, 1999.
      The implementation of the current seismic resistance improvement methods for the existing buildings and structures shall help achieve the following:
      4. improve the preparation of the authorities and residents for major earthquakes;
      5. significantly reduce seismic risks and the consequences of both major earthquakes and industrial disasters caused by earthquakes;
      6. provide the economic and social stability of the country in case of emergencies.

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