

Seismic Isolation Strategies for Earthquake-Resistant Construction

Seismic Isolation Strategies for Earthquake-Resistant Construction:

Emerging Opportunities

By

Mikayel Melkumyan

Cambridge
Scholars
Publishing



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This book first published 2019

Cambridge Scholars Publishing

Lady Stephenson Library, Newcastle upon Tyne, NE6 2PA, UK

British Library Cataloguing in Publication Data

A catalogue record for this book is available from the British Library

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ISBN (10): 1-5275-1802-7

ISBN (13): 978-1-5275-1802-5

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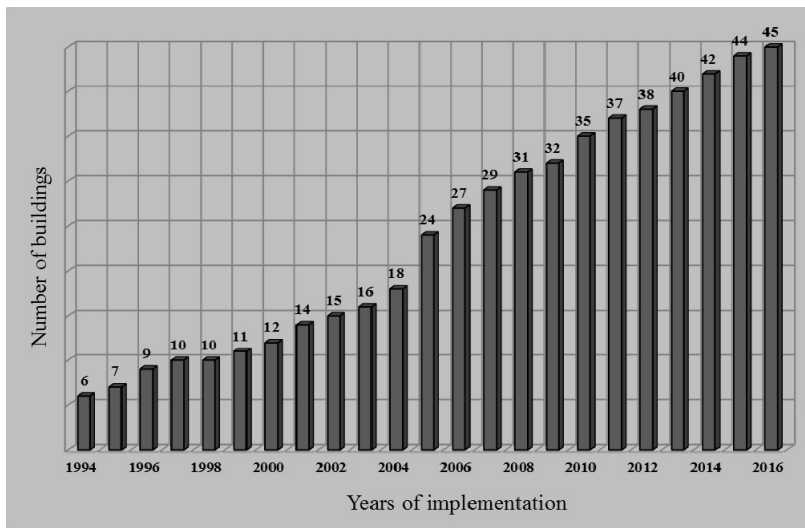
CHAPTER ONE

INTRODUCTION

The protection of existing buildings from earthquake damage is becoming more and more important with every recent seismic event across the world. In this regard, the seismic isolation of structures is becoming an increasingly common method of providing such protection. Particularly, roof isolation techniques forming different types of dampers for earthquake protection, and base isolation technologies for seismic retrofitting, developed by the author of this book, have received a wide application in Armenia. Also, the developed base isolation technology has been implemented in Russia, and a retrofitting design has also been produced for implementation in Romania and approved by the Romanian government. It is worth noting that the above-mentioned seismic (roof and base) isolation strategies have great potential for the rehabilitation of existing civil structures such as apartment blocks and critical facilities such as schools and hospitals. In the case of base isolation, the first dynamic mode of the isolated building involves deformation in the isolation system only; the building above being to all intents and purposes rigid. The higher modes do not participate in the motion, so that the high energy in the ground motion at these higher frequencies cannot be transmitted into the building (Naeim & Kelly, 1999). By reducing the seismic forces transmitted, isolation protects the contents and secondary structural features, as well as the main structure.

Starting in 1995, the author of this book has designed 53 buildings and structures which apply base or roof isolation systems. Of these designed buildings, the total number of already constructed and retrofitted buildings has reached 45 (Figure 1). Among them there are bath-houses, private residences, school buildings, clinics and hospital buildings, business centers, hotels, and apartment buildings.

Figure 1. Number of seismic (base and roof) isolated buildings newly constructed or retrofitted in Armenia by years

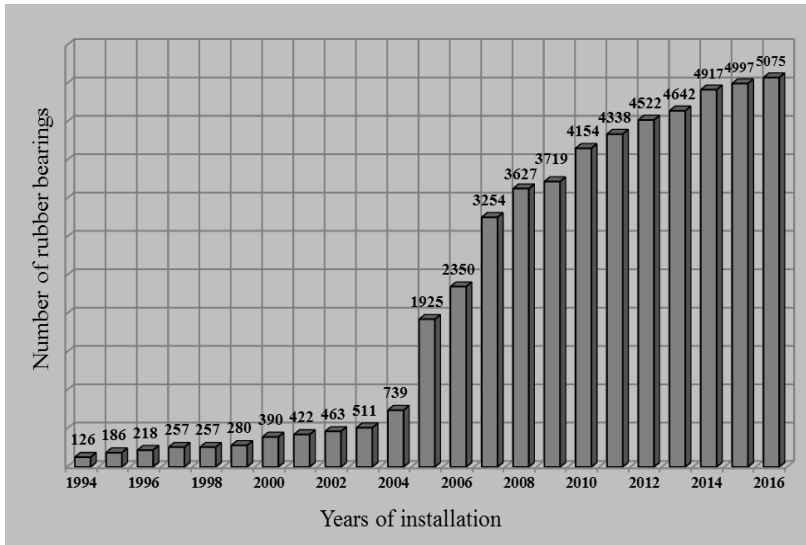


The number of seismically isolated buildings per capita in Armenia is among the highest in the world. Alongside this, seismic isolation laminated rubber-steel bearings (SILRSBs) differing by shape and dimension as well as by damping (low, medium and high) have been designed, and more than 5000 SILRSBs were manufactured in the country, tested locally, and applied in construction (Figure 2).

Thus, by creating the above-mentioned seismic retrofitting technologies and implementing them in different existing buildings the author has accumulated rich experience, which is shared below in the account of several remarkable projects. One of them is the retrofitting by base isolation of a 5-storey stone apartment building (Figure 3a) in Armenia. The operation was made without the resettlement of the occupants (Melkumyan, 1997). World practice provides no similar precedent in the retrofitting of apartment buildings.

The next development of the technology was to utilize the newly developed method of an additional isolated upper floor (AIUF) acting as a vibration damper. This technology of roof isolation was used in earthquake protection design and implemented in two existing reinforced concrete (R/C) 9-storey standard design frame buildings (Figure 3b), also in Armenia (Melkumyan, 2007).

Figure 2. Number of rubber bearings installed in the newly constructed or retrofitted buildings in Armenia by years

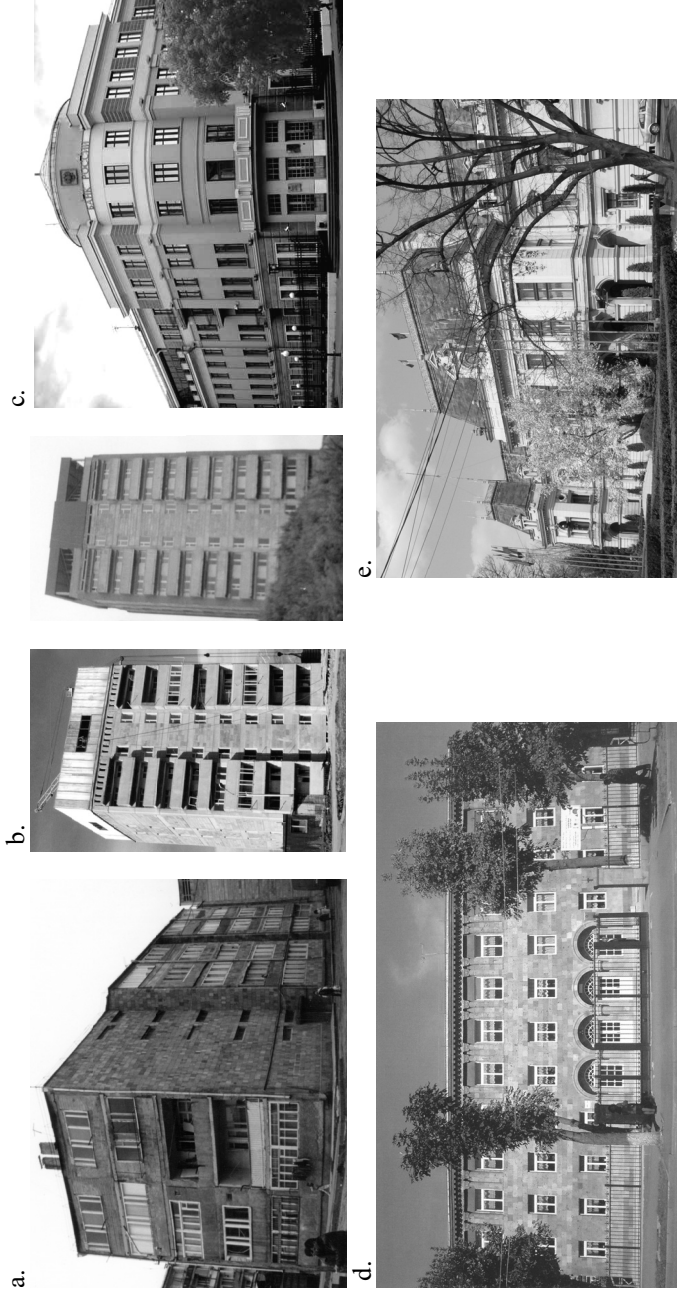


Then, by the end of the nineties, another project initiated by Prof. Eisenberg (Smirnov et al., 2000) to retrofit a 100 year-old 3-storey stone bank building was implemented in Russia with an increase in the building's number of storeys to 4 (Figure 3c). It was emphasized by the project team that to retrofit the existing bank building using base isolation they had implemented the method developed in 1997 by the author (Melkumyan, 1997), who provided Russian colleagues with all the needed drawings, photos and video film related to the retrofitting works already carried out in Armenia.

An additional project to mention is the retrofitting of a 60 years-old non-engineered 3-storey stone school building which has historical importance as well as great architectural value (Figure 3d). Unique operations were carried out in order to install the isolation system within the basement of this building and to preserve its architectural appearance (Melkumyan et al., 2003a, 2003b).

Experience accumulated in Armenia in the retrofitting of existing buildings, including those of historical and architectural value, created a good basis for participation in the international competition announced by the Government of Romania for the development of a design for the retrofitting

Figure 3. Projects on retrofitting by base and roof isolation strategies and implemented in Armenia and Russia or, to be implemented in Romania



of a 180-year old 3-storey historical building (the Iasi City Hall) by base isolation (Figure 3e). The structural concept, including a new approach to installing seismic isolation rubber bearings, was developed and the design of the retrofitting was accomplished in cooperation with the Romanian company, MIHUL S.R.L. (Melkumyan et al., 2011, Melkumyan, 2011).

Based on the experience gained, the new and unique seismic isolation strategies, structural concepts and technologies were developed, providing earthquake safety for the existing buildings (Melkumyan, 2014a). This book consists of two main parts: the first of them is dedicated to the concept of dampers for the earthquake protection of existing buildings, the study of their efficiency, the theoretical background, experimental investigations and implementation in the form of roof isolation and other damper systems; and the second part is dedicated to the concepts of retrofitting by base isolation and their implementation in different types of existing frame and stone buildings.

In the first part of the book, together with the two above-mentioned existing buildings protected by an AIUF, a new structural concept of earthquake protection by roof isolation (Melkumyan, 2014b), designed for application in the existing 12-storey R/C office building erected in 1995, is described. The top floor of this building is an attic, the slab of which had to be separated from the columns, and then lifted up and SILRSBs, with high damping of about 15%, installed between them. There was a need to increase the damping, something that was achieved by a special measure which, together with the sizes and physical/mechanical parameters of the applied SILRSBs, will be presented in this book. Thus, the aforementioned slab is converted into an isolated upper slab which subsequently acts as a tuned mass damper, protecting the building from strong seismic events.

Dampers could also be applied to a base isolated structure in order to restrict an overall displacement of the isolation system. In such a case, the damper must be attached immediately at the level of this system. However, supplementary damping to reduce the displacement demand on the isolation system may increase the in-structure accelerations (Kelly, 1999). In order to avoid this effect a new type of damper to restrict the displacements of seismically isolated buildings called a Dynamic Damper (DD) was developed and proposed by the author (Melkumyan, 2000). The name “dynamic damper” or “dynamic absorber” is applied when the auxiliary mass system has little damping (Reed, 1961). It is suggested to use, as the mass of the DD, the perimeter pavement around the building which is separated from the superstructure and hung to it by means of laminated rubber

bearings. Other types of DD are also considered. The stiffness and mass of the DD should be chosen such that the period of vibration of the DD is equal to that of the isolated building. Such a damper will allow us to decrease the horizontal displacements and also to simplify the isolation system itself. The suggested structural system of the DD also increases the overturning resistance of the isolated building (Melkumyan, 2000, Taniguchi et al., 2008).

In the second part of the book new strategies for retrofitting by base isolation are described. One of these relates to a 55 year-old 4-storey R/C industrial frame building, which had to be simultaneously reconstructed into a 6-storey hotel building (Melkumyan, 2014c). Principally, new structural approaches have been proposed for the seismic isolation of the building's R/C bearing frame, and also innovative new technology for retrofitting by base isolation was developed and applied to this building. SILRSBs with high damping of about 15% were used. However, these bearings are different than those mentioned above for the 12-storey office building. Creative solutions are proposed for the first time, and envisage placing the seismic isolators around the existing columns and then gradually cutting these structural elements. Operations are designed to be performed for the existing columns in several stages.

Further developments have also taken place for retrofitting by base isolation, but in contrast to the above-mentioned frame building, the new structural solution and technology were suggested for an R/C braced-frame building. Unique operations for the retrofitting of the 8-storey Hematology Center Hospital Building (HCHB) (Melkumyan, 2014d, Melkumyan, 2015) constructed about 45 years ago are also described. The seismic isolation of this building is planned to be implemented at the basement level. SILRSBs of the same type as for the 4-storey R/C industrial building were used. Detailed descriptions of all phases for cutting the columns and shear walls, and placing the seismic isolators of the same sizes and physical/mechanical parameters are given in the book.

The second part of the book is also focused on the retrofitting design of an existing stone college building constructed in 1950, and another damaged stone school building constructed in 1959. The structural concepts of retrofitting by base isolation developed for these buildings on the basis of the acquired experience are presented. At the same time, special attention is given to the strengthening, by R/C jackets, of the damaged parts of the existing stone load-bearing walls. It is mentioned that there is no basement in most of the area under the school building, the basement existing only

under a part of the building. Therefore, in order to efficiently implement an isolation system, the creative solution envisages an extension of the basement throughout the whole built surface of the building.

Earthquake response analyses for all the above-mentioned buildings were carried out by the Armenian Seismic Code and acceleration time histories and their detailed results are given in the book.

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CHAPTER TWO

CONCEPTS OF DAMPERS FOR THE EARTHQUAKE PROTECTION OF EXISTING BUILDINGS AND FOR DISPLACEMENT RESTRAINTS IN SEISMICALLY ISOLATED BUILDINGS

2.1. Study of the Efficiency of a Tuned Single and Double Mass Damper on a Model of a Frame Building under Vibration Tests

There are a rather large number of studies concerned with the theoretical investigation of the behavior of buildings with tuned mass dampers under various impacts (Nawrotzki, 2003, Palazzo et al., 2004, Fujitani & Saito, 2006, Taniguchi et al., 2008), however, the experimental studies in this area are quite limited. Melkumyan (1996, 1998, 2007, 2011) has described two works implemented through vibration tests on Series 111 nine-storey full-scale residential buildings equipped with tuned mass dampers in the form of a flexible upper floor (FUF) and an isolated upper floor (IUF) in Vanadzor city, Armenia.

The work presented in this chapter to a certain extent fills the gap in experimental studies investigating the behavior of buildings with tuned mass dampers. The studies have been conducted on a model of the same Series 111 nine-storey frame building. The author designed and made the model out of reinforced concrete on a scale of 1:5 (Melkumyan, 1993), using the principle of simple similarity developed in 1965 (Nazarov, 1965). It had nine columns forming two 120cm spans in mutually perpendicular directions, with axial dimensions of 240x240cm. In one of the directions, the spatial stiffness of the model was provided by three frames with strong load-bearing beams (frame design), and in the other – by three frames with weak binding beams and a single shear wall located in one of the spans in the plane of the middle frame (braced-frame design). The cross-sections of

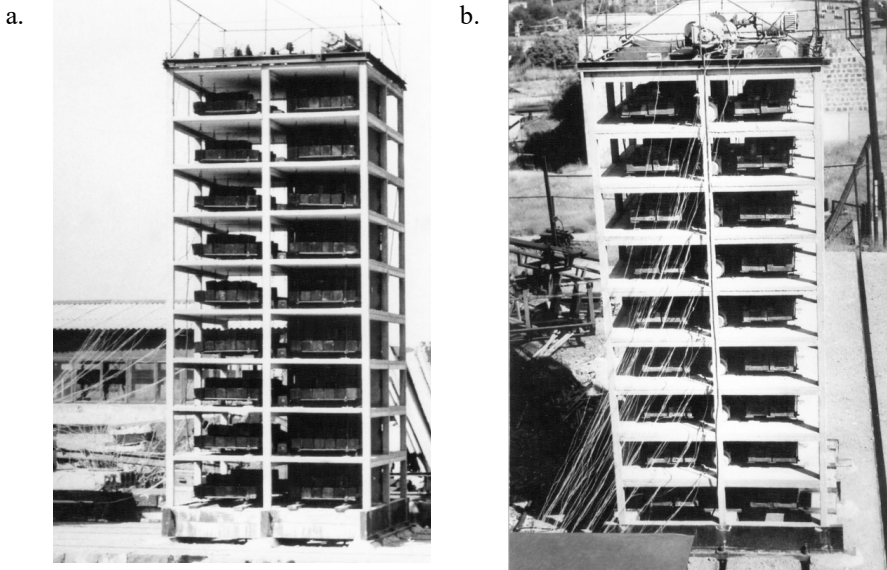
the columns were 8x8cm, those of the bearing beams – 8x10.4(h)cm and the binding beams – 8x5(h)cm. The floors consisted of prefabricated hollow-core model slabs with a thickness of 4.4cm, whereas the prefabricated panels of the shear wall were 2.8cm thick. In accordance with the structural concept of the Series 111 buildings, the shear wall panels were connected to columns by welding the embedded items, and their connection to the beams was provided by casting concrete over the dowels protruding from the shear wall prefab panels. The total height of the model was 5.9m, with each storey being 0.6m high; the foundation beams were 0.4m high and the column steel caps extended beyond the 9th storey slab by 0.1m.

A general view of the model, before the placement of the tuned mass dampers, is shown in Figure 1. As seen, cast iron weights were suspended from the model's slabs to provide vertical loads for creating the necessary level of stress-strain state in the structural elements of the model. More detailed information on the design, construction and testing of this model, without tuned mass dampers, can be found in other publications of the author, and not provided here for brevity.

Before placing the tuned mass dampers, the model vibrations were induced in both directions by laboratory vibration machines specially developed and made by the author for model tests. The vibration machines were placed on the 9th storey's slab and rigidly connected to a steel frame, which in turn was welded to the steel caps of all nine columns of the model (Figure 2). The vibration machines were mounted in such a manner that the exciting forces created during the rotation of the weights eccentrically placed on the vibrators shafts acted on the vertical planes, passing in mutually perpendicular directions through the middle axes (frames) of the model. The revolutions of the vibration machines' DC motors were continuously variable with the help of a special device, thus providing harmonic vibrations in the range from 0 to 10Hz. The parameters of the forced vibrations were measured by large displacement seismographs, as well as accelerographs placed at the level of all floors along the central vertical axis of the model, with the exception of the devices on the 9th floor, where they were placed outside the central zone of the model's plan, as it was occupied by the vibration machines.

Figure 1. Series 111 nine-storey frame building model designed and constructed on a scale of 1:5;

a – front view, b – side view



Vibration tests on the Series 111 nine-storey full-scale frame buildings equipped with tuned mass dampers in the form of FUF or IUF have been mentioned above. Those systems were single mass dampers. Conversely, the subject of this experimental work was the tuned double mass damper. Prior to mounting such damper, as an additional tenth floor, the dynamic characteristics of the model (periods and damping ratios) had been determined. The vibration machine was used to induce forced resonant vibrations of 1st, 2nd and 3rd modes. The values of periods and damping ratios calculated, based on the records of these vibrations, are indicated in Table 1.

Figure 2. Laboratory vibration machines for testing large models, placed on the slab of the 9th storey (before mounting the dampers) with a steel frame rigidly fixed to the steel caps of the nine-storey frame building model columns

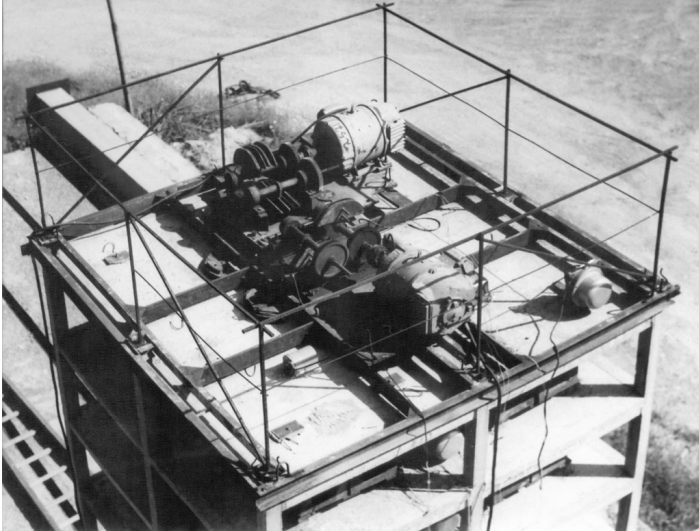


Table 1. Periods of oscillations and damping ratios of the nine-storey frame building model at the stage of elastic behavior under the first three modes of resonant vibrations, prior to mounting the tuned double mass damper

Direction of the tests	Periods of oscillations, sec			Damping ratio ξ , %
	For the 1 st mode T_1	For the 2 nd mode T_2	For the 3 rd mode T_3	
Along the frames with strong bearing beams (frame design)	0.326	0.133	0.075	4.3
Along the frames with weak binding beams and shear wall (braced-frame design)	0.280	0.088	0.049	4.0

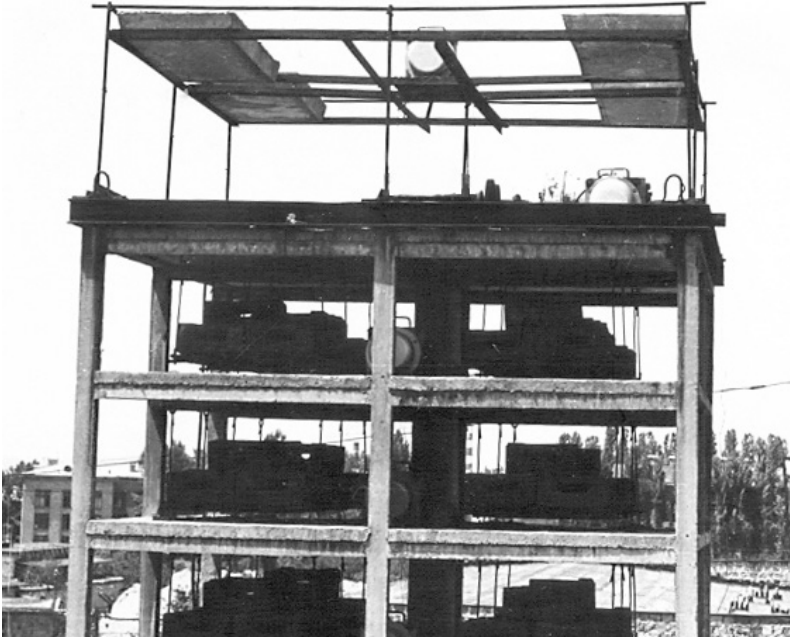
The experimental studies of the nine-storey frame building model with a tuned double mass damper were intended to be carried out under its loading only in the direction of the frames with weak binding beams and the shear wall located in one of the spans of the plane of the middle frame. Given this circumstance, one of the vibration machines was dismantled. In order to reach the cracking stage, in the mentioned direction, the model was subjected to several phases of dynamic loading with a gradual increase in the mass of the off-center weights on the vibrator shafts. In these conditions the shear wall panels suffered light damage in the form of inclined, intersecting thin cracks; whereas the fundamental period of oscillations increased by 1.25 times compared to the initial one and equaled 0.35sec ($\omega_1=2.86\text{Hz}$). Almost no change of the damping ratio was observed ($\xi=4\%$), while the 9th storey's floor displacement amplitude was $A=3.1\text{cm}$.

The obtained values served as a basis for choosing the mass and stiffness parameters of the first damper, which was named as "the main" by the author. As it turned out, the necessary horizontal stiffness of the mentioned additional tenth storey, i.e. the tuned mass damper, could be achieved by using square iron posts with a cross-section of 14x14mm, welded to the sides of the steel caps of the nine-storey frame building model's columns. The stiffness of the model's main damper ceiling could be ensured by 45x45x4mm L-shaped rolled steel profiles, whereas the mass could be accumulated by R/C plates placed on these profiles (Figure 3). Thus, the design of the main damper created on the nine-storey model is similar to that of the FUF, the design of which has been developed by the author, and later implemented and tested with his direct participation (Melkumyan, 1998, Minassian et al., 1993).

These studies were aimed at determining the efficiency and damping properties of the tuned double mass damper as compared to those of a single mass damper. The question of optimal damping in the main damper was not dealt with. The main damper's tuning $f_1^2=f_{MD}^2\omega_1^2$ (with no consideration of damping (Korenev & Reznikov, 1981)) was performed immediately on the damper. Initially, the main damper was tuned to the f_{MD} frequency, very close to the resonance frequency ω_1 of the model's 1st mode oscillations. Afterwards, using incremental changes in the damper's mass, the optimal tuning (Yoshizumi et al., 2003) was determined at which the maximum efficiency of damping was achieved. It turned out that the optimal tuning requires the main damper's mass accumulation up to the value of $m_{MD}=360\text{kg}$, which is half of the model's single storey weight (without kentledge) $m_{storey}=720\text{kg}$. Hence, given that total mass of the

model's nine stories, is $M_M=720 \times 9=6480\text{kg}$, then the relative mass is $v=m_{MD} \div M_M=360 \div 6480=0.056$ (or 5.6%), and the tuning $f_1^2=1.0$.

Figure 3. Fragment of the nine-storey model's upper part with a view of the additional tenth storey, i.e. the main damper together with placed R/C plates that constitute its mass



The approach that takes into account the damping, through the optimization of the parameters for tuned mass dampers, differs from the case for dampers without damping, since the optimal values of tuning and relative viscous (or inelastic) resistance for a given value of v are the ones to be determined. The optimal values of tuning for the hypotheses of viscous resistance or internal inelastic resistance are determined in the same way (Korenev & Reznikov, 1981, Warburton, 1982). In our case the optimal tuning, with consideration of damping, differs from the f_1^2 value and is determined by the following formula:

$$f_{op}^2 = \frac{1}{1+v} = \frac{1}{1.056} = 0.95$$

Once the main damper was tuned, it was subjected to dynamic loading with a variable frequency. Figure 4a shows fragments of vibration records obtained by the seismographic instrumentation at the slab of the model's ninth storey and at the level of main damper's top. It was derived from the obtained records that the maximum oscillation amplitude of the model's upper part with the main damper is 1.05cm, which is almost 3 times less (Makino et al., 2009, Shooshtari & Afzali, 2010) than the comparative oscillation amplitude of the model's upper part without a damper (3.1cm). The maximum oscillation amplitudes of the model's upper part with the damper were approximately the same before and after passing through resonance, and corresponded to frequencies of 2.5Hz and 3.3Hz, which are almost symmetrical in relation to the resonant frequency of the model without a damper (2.86Hz). It has to be noted that before passing through resonance the damper acted in the same phase with the model, whereas after passing through resonance it acted in anti-phase relative to the model. The main damper oscillation amplitude was about 3-4 times larger than the amplitudes of the model's upper part oscillation. The free oscillations of the optimally tuned main damper have also been recorded (Figure 4b). The damping ratio of the main damper determined from this record was $\xi=0.7\%$.

Upon completion of the model testing with the main damper, a second weight was added to its ceiling and hence, the single mass damper was turned into a double mass one. The second mass of the damper could be accumulated by weights placed in a 30x40x10cm metallic box weighing 10kg. The metallic box was suspended from the main damper's ceiling by round steel rods with a diameter of 8mm and length of 30cm. The general view of the second suspended mass is shown in Figure 5.

Figure 4. Fragments of vibration records at the top of the nine-storey model with a single mass damper (1) and at the level of the damper's top (2) under dynamic loading created by vibration machine (a), as well as fragment of the single mass damper's normal mode oscillations record (b)

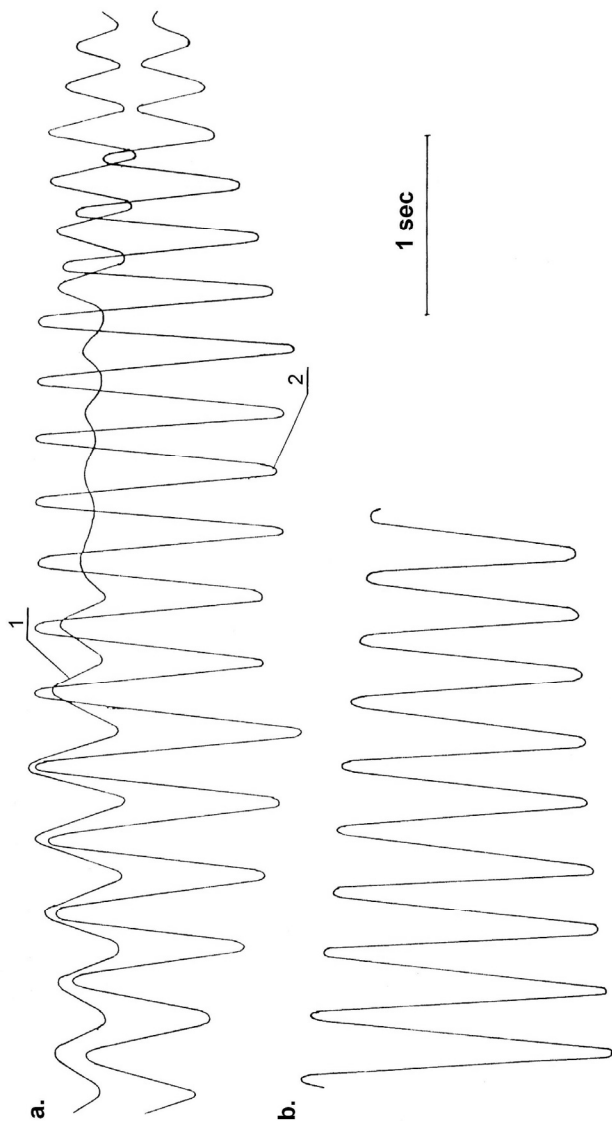
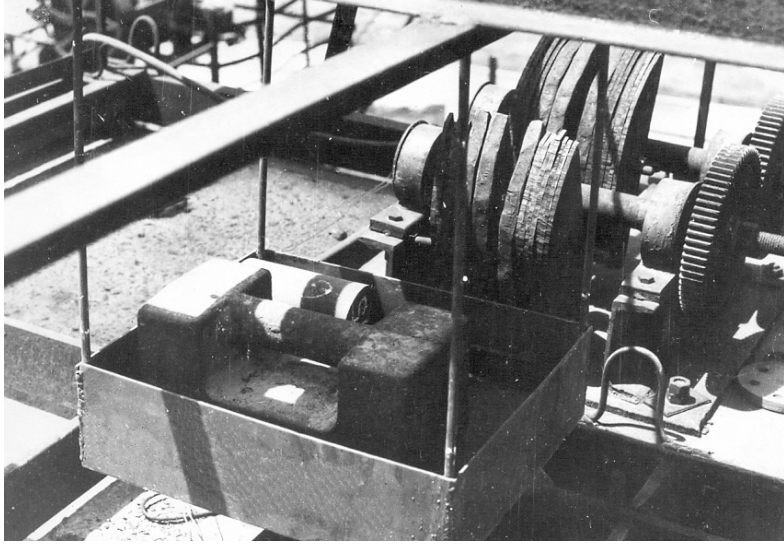


Figure 5. Fragment of the nine-storey model's upper part with the view of the second mass created by weights placed in a metallic box and suspended from the main damper



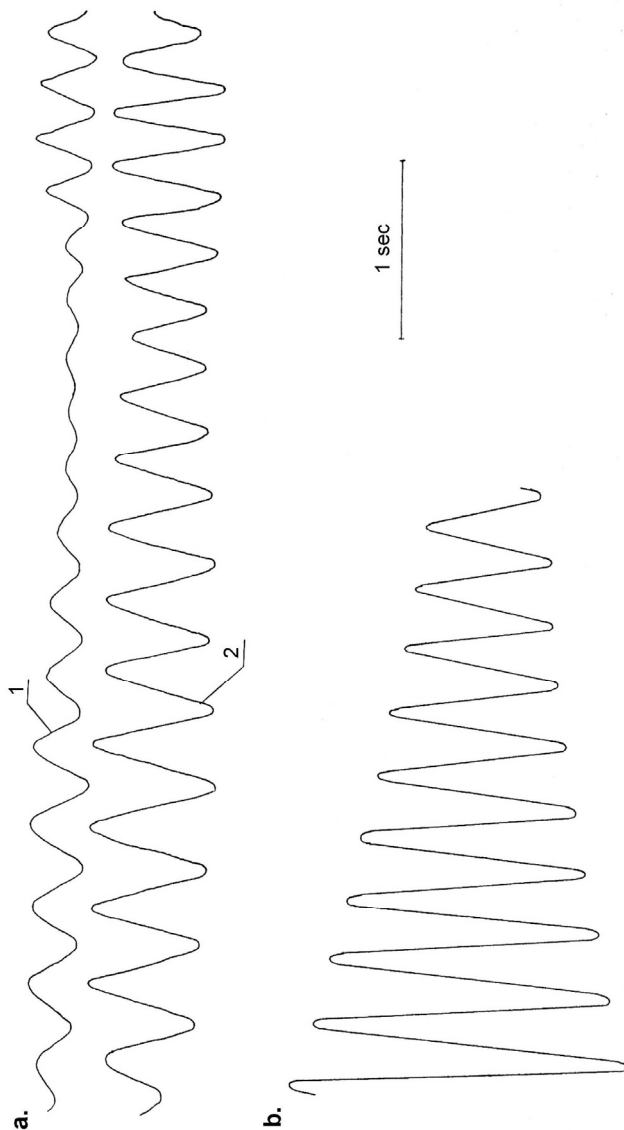
The tuning, or in other words, selection of the optimal value for the second mass was performed in the same manner as the tuning of the main damper. At the optimal tuning the weight for the second mass of the damper turned out to be 40kg, and the partial frequency of its normal mode was 2.65Hz. The relative masses and tunings of the double mass damper were, respectively:

$$\nu_d = \frac{m_{d1} + m_{d2}}{M_M} = \frac{360 + 40}{9 \times 720} = 0.0617;$$

$$f_1^2 = 1.0; \quad f_2^2 = 0.93; \quad \nu_2 = \frac{m_{d2}}{m_{d1}} = \frac{40}{360} = 0.11$$

The total mass of the double mass damper turned out to be 11% higher than that of the single mass damper. Over the course of the tuning of the created damper system, control records of model oscillations with the double mass damper were obtained, again under harmonic impact with variable frequencies. The records of these oscillations are provided in Figure 6. These records, along with the ones shown in Figure 4, were used to

Figure 6. Fragments of vibration records at the top of the nine-storey model with a double mass damper (1) and at the level of the damper's top (2) under dynamic loading created by vibration machine (a), as well as fragment of the double mass damper's normal mode oscillations record (b)

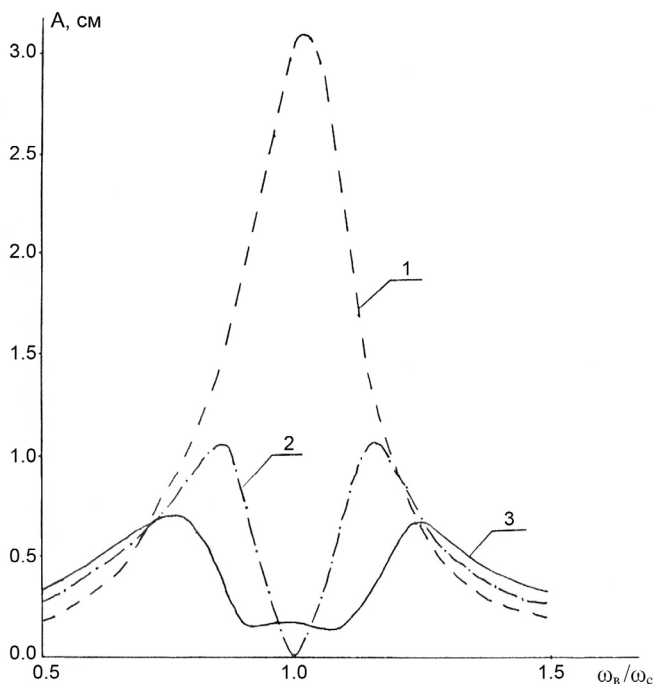


derive the amplitude-frequency characteristics of the model (Figure 7). The obtained results indicate that the tuned double mass damper is more efficient than the single one, since the damping coefficient for the double mass damper reaches 4.4, which is 46.7% higher than that of the single mass damper. Moreover, the range of the frequencies damped is about 50% wider. These results somewhat exceeded our expectations.

Comparing Figures 4 and 6, one may notice that the oscillation amplitudes of the main damper have significantly decreased. Before adding the second mass, the maximum amplitude of the tuned single mass damper was around 3.5cm, whereas after adding the second mass it decreases to 1.4cm. Normal mode oscillation records showed that its damping ratio is $\xi=1.75\%$, i.e. 2.5 times larger as compared to that of the single mass damper. Apparently, the increased damping effect of the tuned double mass damper is explained by the mutual influence of the main damper and the second mass suspended from it, whereby the energy of oscillations is re-distributed. Thus, it was discovered that, under harmonic oscillations induced by a vibration machine, the tuned double mass damper had a higher efficiency than the single one.

Summarizing all of the above, it can be stated that the experimental studies conducted confirm the rather high efficiency of tuned mass dampers, and, thus, that they are undoubtedly worth using to increase the seismic resistance of buildings and structures (Chang et al., 2009, Dorka, 2000). The question of using one or another type of damper depending on the number of masses (single, double or multi-mass), or on structural schemes (FUF or IUF, etc.) at the top of actual buildings is to be addressed. The many factors involved include: the number of storeys, the structural concept and technical condition of the buildings, any circumstances that limit the damper movement, the necessity of utilizing the space inside the damper, the cost effectiveness of the damper, etc.

Figure 7. Amplitude-frequency characteristics of the nine-storey model: (1) without damper; (2) with the single mass damper; and (3) with the double mass damper



2.2. Theoretical Background: Linear and Non-Linear Analyses of a Building with and without TMD

One of the major features of the anti-seismic design of buildings is the possibility of controlling the inertial load values, depending on the structural concept of the buildings. In the 1950s, when the spectral theory of seismic stability was developed, the flexible ground floor was regarded as the basis for reducing the seismic action level. However, the consequences of strong earthquakes such as the 1963 Skopje, the 1988 Spitak or the 2008 Sichuan Earthquakes, etc. have shown that in this case the bearing structures (mainly columns) of the ground floors were severely damaged and further use of buildings was impossible despite the upper floors being well preserved. Therefore, continuing efforts have been made by researchers to discover the most efficient methods of seismic protection for build-

ings and structures, and their practical application. One such method is the TMD, known as a passive vibro-protecting device. A TMD is basically a single-degree-of-freedom appendage of the primary structure (Warburton, 1982). Such dampers have been widely investigated in connection with seismic protection problems (Nawrotzki, 2003, Palazzo et al., 2004, Humar & Wright, 1977, Pinkaew & Lukkunaprasit, 1999).

The natural frequency of a TMD (with the damping neglected) should be equal to the forced vibration frequency of the structure to be protected, which as a rule is represented in the form of a single-degree-of-freedom system. However, during earthquakes, forced vibrations are neither harmonic, nor do they have a preset frequency; while buildings are not single-degree-of-freedom systems. As it is mentioned in Melkumyan (2011), in spite of the chaotic nature of the ground motion, the acceleration time histories of linear oscillators are similar to harmonic vibration processes, with the period equal to that of linear oscillators. Therefore, if the first vibration mode is assumed to be the most significant one during earthquakes, then the natural frequency of the damper should be equal to the first mode frequency of the structure's vibration. When seismic loading is formed due to the superposition of inertial loadings of the first three oscillation modes, then three dampers should be used instead of one, with natural vibration frequencies tuned correspondingly to frequencies of the first three modes of the building's free vibrations.

Thus, in the most general case, the application of several dampers is expedient in terms of reducing the seismic action level in buildings. However, it is technologically difficult to design and make them in the traditional way, that is, in the form of a mass properly fixed to the building. More specifically, even one such damper (for the first vibration mode) in a 10-12 storey building possesses considerable mass, and its practical realization is impossible. Therefore, an additional upper floor for the building has been proposed as a vibration damper (Melkumyan, 2011).

As the mass of the upper floor would be approximately equal to the mass of other floors of the building, this additional floor should exhibit less stiffness than that of the other floors. Thus, a building with a flexible upper floor would be analogous to one with a flexible ground floor. However, there is an essential difference between the two since, after a seismic event, residual deformations in the flexible upper floor are not disastrous for the building as a whole. A flexible upper floor (TMD) could be erected on the existing buildings to increase their seismic resistance, without requiring the tenants to leave the building. Also, a TMD could be widely applied in

new construction.

Determination of the damper parameters essentially depends on the nature of the seismic action. The first attempt to find the optimal parameters of an additional upper tenth floor in a Series 111 nine-storey building with R/C bearing frames and shear walls using accelerograms of various earthquakes is presented below. To simplify the problem, the building design model including the TMD at this first attempt is assumed to be a cantilever beam with masses concentrated at the floor levels. The equations of the forced vibrations of such a system are given by the formula:

$$m_k(\ddot{y}_k + \ddot{y}_0) + a_k(y_k - y_{k-1}) + \mu_k a_k(\dot{y}_k - \dot{y}_{k-1}) - a_{k+1}(y_{k+1} - y_k) - \mu_{k+1} a_{k+1}(\dot{y}_{k+1} - \dot{y}_k) = 0, \\ k = 1, 2, \dots, n; y_0 = 0; a_{n+1} = 0; n = 10,$$

where: m_k , a_k , y_k are the mass, stiffness and displacement of the k^{th} floor of the building, $m_r = m_{10}$, $a_r = a_{10}$, $y_r = y_{10}$ are the mass, stiffness and displacement of the damper (the additional flexible upper 10th floor), $\mu_k = \alpha / \omega$ is the coefficient of viscous damping of the k^{th} floor, and $\ddot{y}_0(t)$ is the ground acceleration (accelerogram).

The values of a floors' stiffness and the mass of the investigated building

are as follows: $a_1 = a_2 = \dots = a_9 = 897000 \text{ kN/m}$; $m_1 = m_2 = \dots = m_8 = 360 \frac{\text{kN} \cdot \text{s}^2}{\text{m}}$;

$m_9 = 430 \frac{\text{kN} \cdot \text{s}^2}{\text{m}}$. Using this data (at $\mu_k = 0$) and the assumed simplified design model, the periods of the first three vibration modes of the building in the direction of shear walls without TMD were obtained: $T_1 = 0.778 \text{ s}$, $T_2 = 0.261 \text{ s}$, $T_3 = 0.159 \text{ s}$. The values of vibration periods of the first three modes for a building with TMD are given in Table 2, where $v = m_r / m_1$, $d = a_r / a_1$ (Melkumyan, 2011).