Paper No. 1338. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

11 WCEE

SEISMIC ISOLATION OF MASONRY BUILDINGS - AN EXPERIMENTAL STUDY

S. NIKOLIC-BRZEV[1] and A. S. ARYA[2]

1 - Sandwell Inc., 1190 Hornby Street, Vancouver, B.C., V6Z 2H6, Canada
2 - Department of Earthquake Engineering, University of Roorkee, Roorkee-247667 (U.P.), India

ABSTRACT

Performance of base isolation technique used for seismic protection of masonry buildings is investigated in the paper. Two-level sliding isolation scheme is explored; this comprises a discrete isolation system in the form of Teflon/Stainless steel sliders at the base level of the building, and a continuous isolation system in the form of "sliding joint" at the floor level. The effectiveness of proposed isolation concept in limiting damage to multi-storey masonry buildings is evaluated experimentally, by testing an one-third scale model of a three-storey high brick building on a shaking table. Relevant aspects of experimental performance of isolated versus fixed-base structure are discussed, such as acceleration amplification ratio, relationship between peak ground acceleration and energy input to the structure, as well as the effect of vertical base excitation. Results of the presented research show that the proposed seismic isolation scheme is capable of reducing dynamic response of multi-storey masonry buildings to high intensity earthquakes appreciably as compared with the conventional system response.

KEYWORDS

Base isolation; buildings; brick masonry; seismic protection; experimental testing; sliding joint.

INTRODUCTION

Pronounced seismic vulnerability of masonry structures was confirmed in numerous past earthquakes worldwide, having as a consequence major devastation of built environment and loss of human lives. To enhance seismic performance of masonry structures, rational strengthening arrangements with emphasis on minimum use of reinforcing steel, have been proposed, verified and incorporated in building Codes and Guidelines worldwide such as IAEE (1986). As an alternative to conventional seismic strengthening methods, base isolation concept emerges as a viable means for reducing vulnerability of both new and existing masonry buildings to strong seismic motions. In the last few decades, base isolation was used in the design of new structures, as well as in seismic upgrading of the existing ones. Field applications of various base isolation systems were preceded by extensive experimental studies. However, only a few research investigations so far were related to base isolation of masonry structures. By and large, such studies treated masonry structures decoupled from the foundations above plinth level by means of a "sliding joint" (Li, 1984; Qamaruddin *et al.*, 1984). This base isolation concept is known as a Pure-Friction (P-F) sliding isolation system. It utilizes friction, allowing some parts of the structure to slide relative to the others in course of an earthquake attack. A review of the P-F base isolation concept was made recently by Nikolic-Brzev (1994).

With only a few exceptions (Paulson *et al.*, 1991; Lou *et al.*, 1992), previous research efforts were directed mainly to base isolation systems for single-storey masonry buildings. When subjected to earthquake-type excitation, such structures demonstrate behaviour characteristic of rigid mass systems. However, due to their inverted mechanical and dynamic characteristics, medium-rise multi-storey masonry buildings subjected to the same excitation respond in a different way, characterized with amplified ground vibrations. Existing large inventory of multi-storey masonry buildings, as well as the need for an innovative strategy for efficient seismic protection of the new ones, necessitate a study on feasible seismic isolation schemes. Results of an extensive experimental and analytical investigation of P-F isolation concept for multi-storey masonry buildings were reported by Nikolic-Brzev (1993), whereas the relevant findings of experimental study are presented in this paper.

OUTLINE OF THE STUDY

The Test Model

To evaluate the effectiveness of proposed isolation scheme in diminishing damage of multi-storey masonry buildings during major earthquakes, performance of both a conventional and isolated model structure was investigated by shake-table testing. Configuration of a model was established to resemble a two room, three-storey high masonry building, with aspect ratio (height/length/width) typical for the Indian housing practice. One door opening in each cross-wall, and four window openings, two by two in each longitudinal wall, were provided in the model, as illustrated in Fig. 1. The model structure was obtained from the prototype one by scaling down all dimensions by a factor of three (1:3 scale). All materials and their characteristics were kept the same. Outside dimensions of the model structure were: length 2.13 m, width 1.65 m, height 3.31 m, and wall thickness 75 mm. Specially manufactured 1:3 scale burnt clay bricks (size 80x40x25 mm, compressive strength 25 N/mm^2), were used. Cement/sand mortar (1:6 ratio) was used for masonry construction. Vertical steel reinforcement bars (5 mm dia, mild steel f_Y = 250 N/mm^2) were provided at the wall corners and junctions, as well as around the openings. One reinforced concrete lintel ring beam (cross-sectional dimension of 30x60 mm) was constructed above the openings at each storey level. Model structure was constructed atop steel plate, that was bolted to the shaking-table before the experiment. Reinforced concrete plinth beam was constructed to simulate foundations and also to provide support for the masonry superstructure. A single model structure was used for evaluating seismic response of both conventional (fixed-base) and isolated masonry structure, thanking to a special arrangement for fixing the base level bond beam to the plinth structure in the initial phase of experiment.

Two different seismic isolation systems were installed at the base and the second storey floor levels respectively. Masonry superstructure was decoupled from the plinth concrete by means of a discrete isolation system comprising a set of Teflon/Stainless Steel sliders (Fig. 2a). In total, 18 sliding bearings were installed at the wall corners and junctions. The upper sliding surface of sliding bearing was made of a stainless steel sheet bonded to a mild steel plate, whereas a lower sliding surface comprised of a steel plate-backed Teflon pad. Components of sliding bearing were bonded to the backup structures by means of epoxy-based adhesive. A "sliding joint" at the second storey level was installed by providing a smoothly finished layer of cement/sand mortar over the floor-slab, and afterwards covering with a thin grease layer (Fig. 2b). Experimentally obtained static frictional coefficient values for Teflon/Steel and grease/concrete sliding couples amounted to 0.1 and 0.4, respectively.

Base Excitation

In total, thirteen shake-table test runs were carried out on a computer-controlled shake-table facility of the Department of Earthquake Engineering, University of Roorkee, India. A single artificially generated earthquake motion of gradually increasing intensity was used throughout the experiment. Target response spectrum curve of simulated earthquake motions used in this study was determined as a smooth broadened envelope curve of the average spectra on rock and stiff soil sites. The test structure was subjected to

Instrumentation

The model was instrumented with ten Force Balance Accelerometers (FBA) during the experiment. For recording horizontal and vertical components of simulated earthquake excitation, two FBAs were mounted on each floor-slab and at the shaking-table level. To evaluate response accelerations at the location of isolation systems, one horizontal FBA was mounted on the bond beam at the ground and second storey levels respectively. Data was sampled at 100 points per second, and 2048 samples per channel were stored in digital form in the course of each table run for further processing. Unfortunately, displacement transducers (LVDTs) required for measuring displacement time histories were not available; however, maximum and residual displacements at the base level were recorded after each shaking-table test run. Finely ground turmeric powder was spread over the lower sliding plate, and displacement values were obtained by measuring steel plate area free of powder after the test.

RESULTS OF EXPERIMENTAL INVESTIGATION

Simulated Earthquake Motion

Characteristics of recorded base motions in each table run are summarized in Table 1. During the initial series of six runs, the model was tested in fixed-base condition. The recorded maximum amplitude of horizontal table motion, i.e. Peak Ground Acceleration (PGA) varied from 0.125g in run No. 1 to 0.207g in run No. 5; vertical component of the base motion was introduced in runs No. 4 and 5, with recorded PGA levels of 0.07g and 0.14g respectively. The isolated model structure was tested in seven test runs, with horizontal table PGA values ranging from 0.201g in run No. 7 to 0.379g in run No. 13; vertical component of the base motion was introduced in runs No. 10 to 13, with PGAs ranging from 0.09g to 0.186g. Due to the feedback of the model and table interaction, it was not possible to generate exactly the same base motion in different table runs. However, peak intensities and acceleration response spectrum curves of horizontal base motions recorded in runs No. 3 (fixed-base) and No. 7 (isolated) were found to be similar. Out of all test runs with bi-directional (i.e. horizontal and vertical) excitation, similar intensities were recorded in runs No. 5 (fixed-base) and No. 10 (isolated), as illustrated in linear response acceleration curves shown in Fig. 3. Also shown are recorded values of roof level accelerations and corresponding fundamental periods inferred from the Fourier amplitude spectra. The results recorded in those four runs were therefore used as a reference for comparison between the seismic response parameters related to fixed-base and isolated model structures respectively.

Dynamic Response Parameters

Calculated peak values of relevant seismic response parameters are summarized in Table 1. It should be noted that maximum values of certain parameters might not have necessarily occurred simultaneously. The effect of phase lag to model response has been neglected in this study. In the course of experiment maximum response accelerations of 0.527g and 0.495g were recorded at the roof level in the fixed-base and isolated model respectively. During the last test run, the isolated structure was subjected to excitation with a recorded horizontal PGA level of 0.4*g*; even at such a high excitation level, the structure had not experienced any structural distress. The fixed-base structure was subjected to peak table excitation of approximately 0.21*g* without any signs of damage.

<u>Base shear forces</u>, expressed as a fraction of the total model weight, were computed from the acceleration records. Total weights of the fixed-base and isolated model amounted to 3.65 and 4.2 tons respectively. Maximum base shear forces of 0.79W (run No. 5) and 1.63W (run No. 13) were recorded in the fixed-base and isolated models respectively. A considerable reduction in maximum base shear forces by around 40% was reported in the isolated structure (run No.10) as compared to the fixed-base one (run No.5). Similar results were obtained by comparing the maximum base shear forces in the runs No.3 (fixed-base) and No.7

(isolated structure), as illustrated in Fig.4.

Maximum and residual sliding displacements at the plinth and second storey floor levels respectively were measured after each test run. The second storey isolation system was not initiated to slide even at a very high base excitation level of around 0.4g (run No. 13). Obviously, maximum response acceleration of 0.27g attained at the second storey level was not sufficient to induce sliding movement of the third storey superstructure.

Table 1. Measured Dynamic Response Parameters

		Run No.												
Run No.		1	2	3	4	5	6	7	8	9	10	11	12	13
Condition		F	F	F	F	F	F	S	S	S	S	S	S	S
Horizont.	ahg (g)	0.13	0.16	0.21	0.12	0.21	0.14	0.20	0.25	0.27	0.21	0.23	0.22	0.38
Accel.	ah3(g)	0.20	0.28	0.49	0.28	0.53	0.28	0.36	0.43	0.51	0.39	0.49	0.33	0.50
	ah3/ahg	1.56	1.76	2.39	2.31	2.51	2.04	1.81	1.73	1.90	1.87	2.14	1.48	1.31
Vertical	avg (g)	-	-	-	0.07	0.14	-	-	-	-	0.15	0.17	0.09	0.19
Accel.	av3(g)	-	-	-	0.20	0.19	-	-	-	-	0.23	0.24	0.22	0.27
	av3/avg	-	-	-	2.90	1.34	-	-	-	-	1.56	1.42	2.45	1.43
Base Shear	Vb/W	0.91	0.68	0.65	0.49	0.79	0.53	0.59	0.62	0.65	0.50	-	1.10	1.63
Residual	Displ(mm)	-	-	-	-	-	-	10	35	82	33	N.A.	N.A.	N.A.

Note: F= fixed-base; S= sliding; a_{hg}= peak horizontal ground acceleration; a_{h3}= max. top storey horizontal response acceleration; a_{h3}/a_{hg}= horizontal accel. amplification ratio; a_{vg}= peak vertical ground acceleration; a_{v3}= max. top storey vertical response acceleration; a_{v3}/a_{vg}= vertical accel. amplification ratio; V_b= max. base shear; W= total model weight.

Free vibration test of the fixed-base model was carried out before the experiment in order to determine its dynamic characteristics, particularly natural frequency and damping ratio. The test was performed by pulling the model in longitudinal direction with the rope attached to the roof floor-slab. The rope was suddenly released and the resulting vibrations were recorded by means of a FBA mounted on the roof floor-slab. The Fourier amplitude spectrum analysis of recorded transient vibrations indicates that the first- and second-mode frequencies were around 5.5 and 10.2 Hz respectively. On the other hand, Fourier amplitude spectra computed using the roof accelerations recorded during the first series of six runs indicate that the first-mode frequency of fixed-base model was in the range from 4.8 to 6.4 Hz. The first-mode frequencies of isolated model structure determined from the Fourier spectrum analysis of recorded roof accelerations indicate a reduction in the first-mode frequency value by 45 to 50% as compared to that of the fixed-base model. Structural damping ratio of 2.37% was determined from the free vibration acceleration record by logarithmic decrement method. The obtained value is comparable with the average damping ratio value of 2.6%, that was determined from the transient vibration records of reference masonry piers constructed simultaneously with the model structure using the same building material.

Acceleration amplification is defined here as a ratio between response acceleration at a certain level of the structure (usually the uppermost level) and the PGA. Experimentally obtained values of the horizontal acceleration amplification ratio were in the range from 1.31 to 2.51. Comparison was made between the amplification ratios of the fixed-base system (run No.3) and the isolated one (run No.7) subjected to similar excitation level (PGA of approximately 0.2*g*). Figure 5a shows that the amplification of horizontal accelerations was considerably higher in the fixed-base structure (run No.3) than in the isolated one (run No.7). Reduction in the acceleration response of isolated structure as compared to the fixed-base one was not as remarkable as expected. However, it was observed that the minimum value of amplification ratio (1.31) was obtained during the last test run, when the model was subjected to the highest ever excitation of approximately 0.4*g*. Further on, a comparative study on the system responses during test runs with increasing excitation levels was carried out. It was shown that efficiency of the proposed isolation scheme in terms of reduction in acceleration responses was more pronounced at the higher excitations.

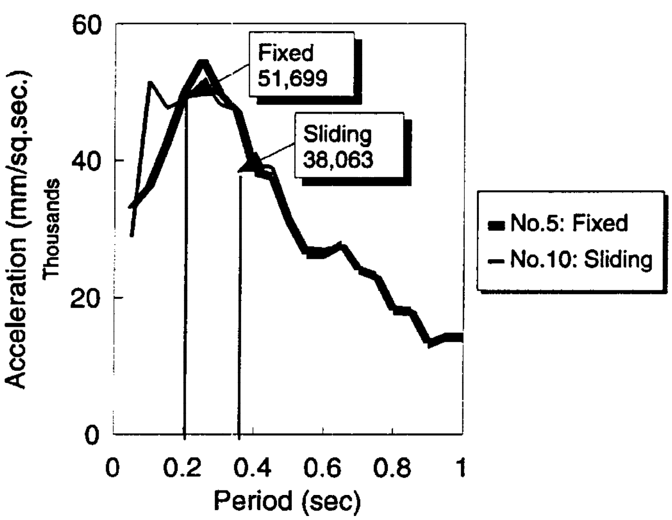

Fig. 3. Acceleration response spectra for measured base motions- Runs No.5 (fixed-base) and No.10 (sliding structure).

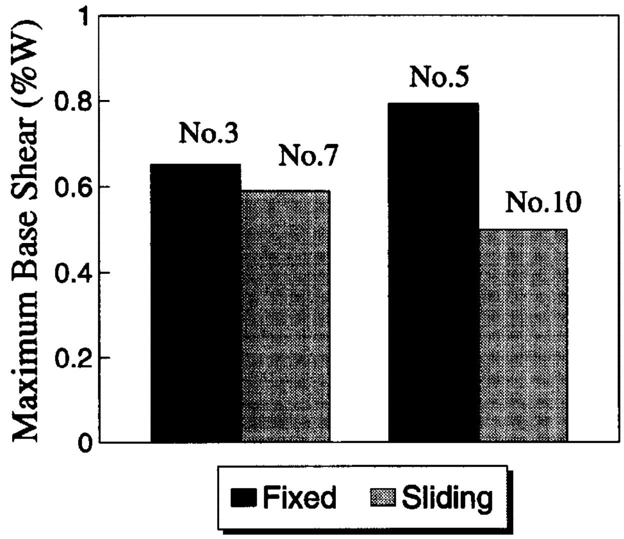

Fig. 4. Comparison of maximum base shear forces in the fixed-base (runs No.3 and 5) and sliding structure (runs No.5 and 10).

Effect of vertical base motion to dynamic response of isolated structure was compared in the test runs No. 7 and 10; those two runs were characterized with similar horizontal PGA levels of around 0.2g. Although vertical component of the base motion was introduced in the run No.10, horizontal acceleration amplification ratio remained unchanged (see Table 1). Mean values of maximum vertical and horizontal acceleration amplification ratios calculated for all test runs are presented in Fig. 5b. Similar levels of acceleration amplifications obtained in case of models subjected to horizontal and/or vertical base motions indicate that seismic response of a sliding structure is not significantly affected by vertical base motion.

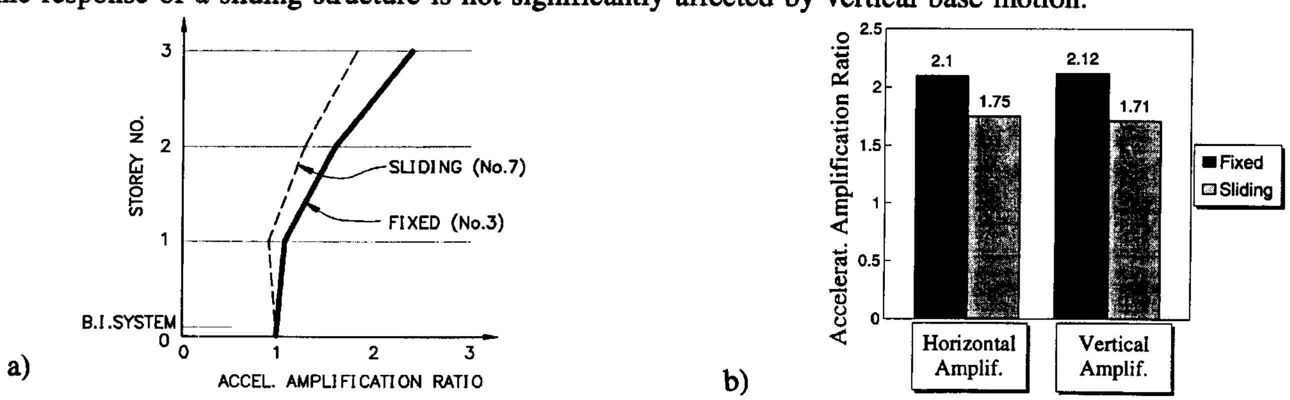

Fig. 5. Dynamic acceleration amplification ratios: a) Horizontal amplification ratio in the fixed-base (run No. 3) and sliding structure (run No. 7), and b) Mean values of horizontal and vertical amplification ratios.

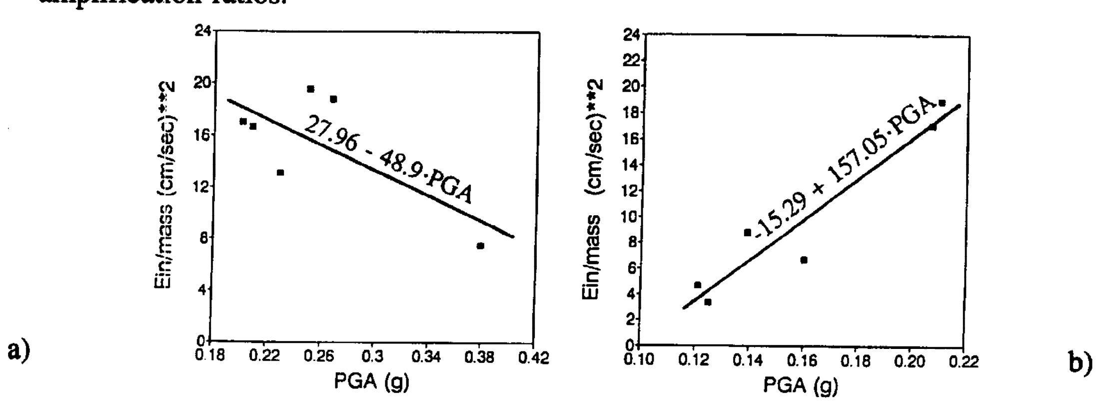

Fig. 6. Earthquake input energy versus PGA level: a) Sliding structure, and b) Fixed-base structure.

Amount of input energy transferred to the structure in the course of simulated earthquake motion is an important parameter related to the effectiveness of a seismic isolation scheme. Response accelerations recorded during the experiment served as a basis for a study on the relationship between PGA and energy input to the model structure. Relevant results of statistical regression analysis are presented in Fig. 6. The study has shown that whereas energy input increased with increasing PGA level in the fixed-base system (Fig. 6b), the reverse trend was observed in the isolated structure (Fig. 6a). This finding is in support of the previous observations concerning the pronounçed efficiency of the P-F isolation scheme at higher levels of base motion.

CONCLUSIONS

As a result of the study presented in this paper, the following conclusions can be drawn:

1. An average reduction in maximum response accelerations by around 30% has been reported in isolated model as compared to the fixed-base one. However, it was observed that the minimum acceleration amplification ratio of 1.31 was obtained in the isolated structure during the test run characterized with the highest ever excitation level of approximately 0.4*g*. Efficiency of the proposed isolation scheme thus becomes more pronounced at the higher excitation levels.
2. The model structure was exposed to simultaneous effects of horizontal and vertical base motions in several test runs. Similar levels of acceleration amplification ratios were attained in test runs with horizontal and vertical base excitations; this indicates that seismic response of the isolated structure is not significantly affected by vertical base excitation.
3. A considerable reduction in maximum base shear forces by around 40% was reported in the isolated model structure as compared to the fixed-base one.
4. A comparative analysis of the amount of earthquake energy input to the fixed-base and sliding system was carried out. The study revealed that the amount of input energy in a fixed-base system is constantly increasing with the increase of horizontal excitation, whereas for a sliding system an opposite trend is observed. This finding is in support of the previous notion related to the increased effectiveness of the proposed isolation scheme at higher excitation levels.

It can be finally concluded that the proposed seismic isolation scheme is capable of reducing dynamic response of multi-storey masonry buildings to high intensity earthquakes appreciably as compared with the fixed-base structures. This scheme may be therefore recommended for adoption as a feasible practical means for seismic protection of low- and medium-rise masonry buildings.

REFERENCES

IAEE 1986. Basic concepts of seismic Codes - Non-engineered construction. *International Association for Earthquake Engineering*. Tokyo.

Nikolic-Brzev, S. (1994). An innovative seismic protection scheme for masonry buildings. In: *Proceedings of the Tenth International Conference on Brick/Block Masonry.*, Vol.1, pp. 273-282. Calgary.

Nikolic-Brzev, S. (1993). Seismic protection of multi-storey brick buildings by seismic isolation technique. *Thesis submitted in the fulfillment of the degree of Doctor of Philosophy*. Department of Earthquake Engineering, University of Roorkee, India.

Paulson, T.J., Abrams,D.P., and Meyes, R.L. (1991). Shaking-table study of base isolation for masonry buildings. *Journal of Structural Engineering ASCE*, 117, 3315-3336.

Qamaruddin, M., Chandra, B., and Arya,A.,S. (1984). Dynamic testing of brick building models. *Proceedings, The Institution of Civil Engineers*, 77, 353-365.

Li, L. (1984). Base isolation measure for aseismic buildings in China. In: *Proceedings of the Eight World Conference on Earthquake Engineering.*, Vol. 6, pp. 791-198. San Francisco.