ISET Journal of Earthquake Technology, Paper No. 376, Vol. 35, No. 4, December 1998, pp. 77-93

A STATE-OF-THE-ART REVIEW OF SEISMIC ISOLATION SCHEME FOR MASONRY BUILDINGS

M. QAMARUDDIN
College of Engineering, Sultan Qaboos University,
P.O. Box. 33, Al-Khod 123,
Sultanate of Oman

ABSTRACT

The concept of base isolation system as a means of seismic protection to structures seems to be more than a century old; but most of the older base isolation systems were unacceptably complicated. Masonry buildings of conventional construction are ideal candidates to be economically isolated by the well-accepted modern base isolation techniques. The concept of friction seismic isolation (FSI) for masonry buildings has been developed as an alternative seismic isolation system. The paper presents a review of worldwide development of a FSI scheme in which isolation mechanism is purely sliding friction for masonry buildings. In the review, the findings concerning the response of structures with sliding support to two horizontal harmonic ground motion components is also presented. In view of such investigations, demonstration brick buildings have been built in China using FSI scheme. Thus, for low-cost masonry buildings, the FSI scheme is an attractive approach.

KEYWORDS: Seismic Base Isolation, Masonry Buildings, Sliding Joint, Coefficient of Friction, Bi-Directional Harmonic Ground Motion

INTRODUCTION

Present earthquake resistant design provisions (Earthquake Resistant Regulations 1992) are based on the assumption of inelastic behavior of structures in major earthquake occurrence. It is generally possible to achieve ductile behavior with significant damage in steel and reinforced concrete structures. However, for masonry that is a brittle material, this approach does not yield satisfactory seismic performance. The conventional masonry buildings have very poor safety records during past earthquakes. Experience gained from occurrence of past earthquakes has shown (Arya et al. 1977, Monge 1969 and Steinbrugge 1963) that masonry buildings have suffered the maximum damage and have accounted for the most loss of life, compared to all other building systems. Much research investigation has been conducted in this area during the last forty years, and simple strengthening measures for masonry buildings have been recommended (Arya 1967, Earthquake Resistant Regulations 1992, Krishna and Chandra 1969, Moinfar 1972, Plummer and Blume 1953 and Yorkdale 1970) for achieving non-collapse masonry constructions. Observations made after earthquakes in recent past in China, India, Iran, Japan, Turkey, USA and former USSR showed that the masonry buildings did not perform satisfactorily during severe earthquake ground motions.

Masonry constructions are increasingly being used in all countries, except perhaps in the relatively more developed ones. A number of base isolation schemes have been developed and implemented (Beck and Skinner 1974, Blakeley et al. 1979, Buckle et al. 1990, Caspe 1970, Chopra et al. 1973, Fintel and Khan 1969, Kelly 1984, Megget 1978, Skinner et al. 1975, Skinner and McVerry 1975, and Skinner et al. 1980) in the last forty years in which a superstructure is connected to a substructure through flexible structural elements and/or energy dissipation devices. The base isolated structures are supported by horizontally flexible but vertically rigid bearings interposed between the base of the structure and its foundation. Nowadays, following base isolation systems (Jangid and Datta 1995) are being used worldwide:

- Lead-rubber bearing system
- Natural rubber bearing system
- High damping rubber bearing system
- Resilient-friction base isolation system
- Sliding resilient-friction system
- Friction pendulum system

It appears that it is difficult to apply the above seismic isolation systems economically to masonry buildings in developing countries specifically in view of special techniques required in their construction. However, friction seismic isolation (FSI) scheme developed during the last twenty years may be employed in masonry buildings economically. FSI system utilizes friction, allowing some parts of a structure to slide relative to the others. In such a system there is no restoring force provided by any type of external horizontal springs. A state-of-the-art review of the FSI scheme for masonry buildings is presented here. The review briefly covers the theoretical and parametric studies conducted to understand the behavior of friction base isolated masonry buildings. The paper specifically addresses the investigations that have been carried out recently and not covered in earlier reviews. Results of some important experimental studies are also included.

CONCEPT OF FRICTION SEISMIC ISOLATION

Base isolation is a design approach in which a structure is protected from the earthquake force by a mechanism, which reduces the transmission of accelerations due to ground motion into the structure. The concept of base isolation technique is to reduce the fundamental frequency of a structure to a value lower than the predominant energy containing frequencies of earthquake ground shaking. The other objective of a base isolation system is to provide a means of energy dissipation to reduce the energy, generated by ground motion, which could be transmitted to the superstructure.

Seismic base isolation consists of de-coupling the structure from the damaging effect of horizontal component of the earthquake ground motion. The main idea for solving the seismic strength problem of masonry buildings has come from the past history of earthquakes. After the Dhubri earthquake, the damage study (Gee 1934) showed that those buildings in which the possibility of movement existed between the superstructure and substructure suffered less damage than those buildings in which no such freedom existed. Based on such encouraging seismic behavior of small structures, a simple mathematical model was introduced by Qamaruddin (1978) and Arya et al. (1978) to compute the seismic response of masonry building with friction base isolation system. In such buildings, a clear smoothened surface is created at plinth level, on which the superstructure simply rests and is free to slide except for frictional resistance. This discontinuity at the plinth level enables the building to dissipate a part of the seismic energy by sliding. The FSI concept was established through analytical and experimental studies (Qamaruddin, 1978) made for masonry buildings.

The concept of FSI system was further strengthened by the damage studies made (Li 1984) after the Xintai (1966), Bohai (1969) and Tangshan (1976) earthquakes in which it was found that adobe buildings which were free to slide on their foundations (by accident) survived with little or no damage whereas others which were tied on their foundations collapsed. Experimental and theoretical studies have been made by researchers to incorporate such a friction seismic isolation system in masonry buildings to achieve a collapse free if not a damage free performance during the earthquakes. In the following paragraphs, both types of studies are briefly described.

EXPERIMENTAL STUDIES

Experimental investigations, which were carried out to study the dynamic and seismic response of the masonry structural models, are briefly discussed in the following sections to show the effectiveness of the FSI concept.

1. Tests on One-Fourth Scale Single Story House Models

Feasibility study of the friction seismic isolation concept was made by testing (Qamaruddin 1978, Arya et al. 1978 and Qamaruddin et al. 1984) pilot house models, by inserting different sliding materials (such as graphite powder, dry sand and wet sand) between the house models and their base. The observed coefficients of friction (F) were 0.25 for graphite powder, 0.34 for dry sand and 0.41 for wet sand. It was observed by steady state testing of the pilot models that in the case of base isolated models there were no amplification of accelerations, whereas such acceleration amplification was observed in similar fixed-base structures. Using graphite powder as a sliding material, the ratio of top/base acceleration was nearly equal to 0.63 which, for the same model fixed at the base, was as high as 2.34. Thus, the pilot tests on base isolated models strengthened the base isolation concept that by introducing a discontinuity at the plinth level of the superstructure, the effective seismic force can be reduced as compared to that of the conventional models.

2. Dynamic Load Tests on Single Story House Models

The experimental investigations were carried out by the researchers to study the dynamic behavior and to compare the performance of the FSI house models with that of the conventional ones. These models were fabricated with different seismic schemes and subjected to (a) shock load, and (b) blast induced strong ground motion.

Small Scale Models: Eight half-scale models of a typical brick house were tested (Qamaruddin 1978, Arya et al. 1978 and Qamaruddin et al. 1984) under shock loading. Out of eight models, there were six conventional ones, and two models with a sliding substructure (Figure 1). Besides the features of fixity or sliding at plinth level of model, the quality of mortar, wall thickness and reinforcing patterns were also varied in the house models. The tests on these models were performed in two sets of four model houses each on a specially fabricated railway wagon shock table facility (Qamaruddin 1978). The houses were tested to ultimate state through gradually increased shocks. Sliding joint was provided between the superstructure bond beam and the plinth band by using burnt mobil oil as a sliding layer.

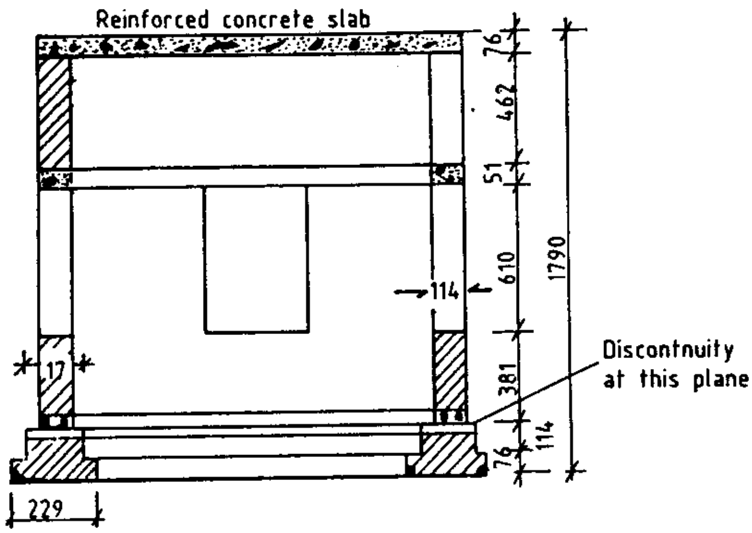

Fig. 1 Brick house model with sliding base

The tests results showed that the house models with sliding base had a significant reduction in response and exhibited adequate behavior up to very high base accelerations as in comparison with to the similar models with fixed base (Qamaruddin 1978). In sliding house models, it was also observed that the roof acceleration showed approximately a fixed value after the shock was severe enough to cause cracking. On the contrary, the roof acceleration continued to build up until sufficient cracking occurred in the fixed base model structures. The extent of cracking was much less in sliding models than in fixed base ones. The shear walls were severely damaged in fixed base structures, but in sliding type models, the cross walls were more damaged. Also, whereas the shear walls developed mainly diagonal cracks, the cross walls damaged with horizontal cracks. The sliding displacement of sliding models was found reversible in reversed shocks. The residual displacement was thus reduced after each complete cycle. The house model in mud mortar with sliding base also showed extremely good shock resistance and it could withstand without collapse large base shocks, though with more severe damage than the similar model in cement mortar (Qamaruddin 1978).

Similar FSI concept has been proposed and tested in China (Li 1984; Fu 1988). Upon concluding that existing base isolation systems are not economically feasible for the low-cost housing in developing countries such as China, a very simple isolation system was proposed (Li 1984). The system consisted of laying a specially screened sand layer between terrazzo plates on the base floor level. Dynamic frictional coefficient of rolling sand was found to be 0.2. Li (1984) tested five small-scale house models on shake table to examine the effectiveness of FSI system. All the five house models survived the shaking of the table and a little sliding of the superstructure was observed at 0.2g acceleration of the table. Model studies were also carried on aluminum and masonry specimens with FSI scheme (Zongjin et al. 1989). Four series of tests were performed with fixed-base of the structure fixed, followed by one series of tests on base isolated models using three sliding materials. Figure 2 shows a comparison of acceleration spectrum obtained from experimental response and theoretical response studies for both the fixed-base and the sliding base structures. There is reasonably good agreement between the experimental and theoretical results for the sliding structure.

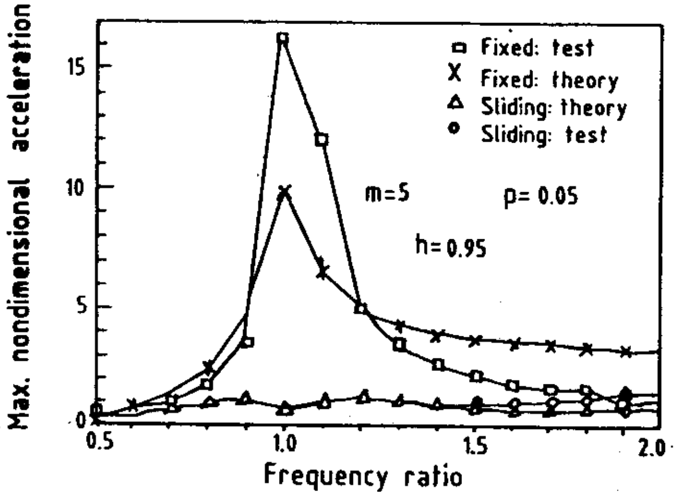

Fig. 2 Comparison of acceleration spectrum values

Lou et al. (1992) made horizontal load tests on brick walls with and without sliding joint under various vertical loads. It was observed from the test results that the horizontal load of the conventional wall, when the formation of cracks started was two times larger in comparison with the horizontal load resisted by the wall with sliding joint. Shaking table tests were also made (Lou et al. 1992) with various sliding joint specimens. Graphite powder, asphalt felt, screened gravel or fine sand and paraffin wax were employed as sliding material. The test results showed that the graphite powder was an ideal low friction material to make sliding joint.

Testing of Full-Scale Model: One full-scale FSI house model, constructed with under-fired bricks of low strength, was tested under blast-induced loading, equivalent to high intensity earthquake (Li 1984). The house model survived the blast loading but shifted a little distance at the base.

3. Shaking Table Test on Multistory House Models

Bingze et al. (1990) carried out testing of two six-story reduced scale gypsum models on shaking table to investigate dynamic behavior of base isolated multistory brick buildings. One of the models had FSI scheme (model 1) whereas the other model was fixed directly on the shaking table (model 2). The FSI system was composed of upper and lower reinforced concrete ring beams filled with two layers of asphalt felt in which graphite powder was interposed. The building models were excited on the shaking table with input acceleration varying from 0.1g to 0.5g. Sliding was initiated at 0.25g acceleration in model 1. No damage was seen in model 1 up to 0.5g base acceleration of the table. The model 2 collapsed at the end of the test.

Nikolic-Brzev and Arya (1990) and Nikolic-Brzev (1994) have made a review of the pure-friction (P-F) base isolation concept. It turned out from the review that previous research efforts were directed mainly to base isolation systems for single strorey masonry buildings with only a few exceptions (Bingze et al., 1990; Paulson et al., 1991). Under earthquake-type excitations, dynamic characteristics of such structures are similar to rigid mass systems. But, due to their inverted mechanical and dynamic characteristics, medium-rise multistorey buildings subject to the same excitation respond in a different way, characterized by amplified ground vibrations. In view of a large number of existing multistorey masonry buildings and the need for an innovative strategy for efficient seismic protection of the new ones, Nikolic-Brzev and Arya (1996) reported the relevant findings of extensive experimental investigation of P-F isolation concept for multistorey masonry buildings. Performance of both a conventional and isolated model of a three storey high brick building (one-third scale) was investigated by a shake table testing. Different seismic isolation systems (Nikolic-Brzev and Arya, 1996) were installed at the base level (a set of teflon/stainless steel sliders) and the second storey floor level (grease/concrete sliding joint). Experimentally obtained static frictional coefficient values for teflon/steel and grease/concrete sliding joints amounted to 0.1 and 0.4, respectively.

Thirteen shake table test runs were carried out on a computer-controlled shake table facility (Nikolic-Brzev and Arya, 1996). A single artificially generated earthquake motion was used throughout the experimental investigation. The test structure was subjected to simultaneous effects of horizontal and vertical base motions in the course of several test runs. The model structure was tested in the fixed base condition in the first series of six test runs by keeping the peak ground acceleration in the range of 0.2g. Input excitation level was increased during the second series of seven test runs on sliding structure with maximum peak ground acceleration level of 0.4g in the last run. It turns out from the results of this investigation that similar levels of acceleration amplifications ratios were attained in test runs with horizontal and horizontal and vertical base excitations indicating that the seismic response of the isolated structure is not significantly affected by vertical base excitation. The results also indicate that an average reduction in maximum acceleration response by around 30% was obtained in isolated structure as compared to the fixed base one. A reduction of around 40% in maximum base shear was reported in the isolated model structure as compared to the fixed base model. This investigation revealed that the amount of input energy in a fixed base structure was increasing with the increase of horizontal excitation, whereas an opposite trend was observed for a sliding structure. Finally, it is concluded from this study (Nikolic-Brzev and Arya, 1996) that the isolation scheme is capable of reducing seismic response of multistorey masonry buildings to high intensity earthquakes appreciably as compared with the fixed base structure.

4. Construction of Experimental Buildings with FSI Scheme

Four buildings with the FSI measure were built in China (Li 1984). One 16 m^2 adobe house with straw roof was constructed at Huaping County (Yunnan Province) in 1975. Another 16 m^2 house with tamped wall and tile roof was built at Xichang city (Sichuan Province) in 1975. In 1980, a 12 m^2 house was built with brick walls and tile roof at Anyang City of Henan Province. A four-story dormitory was constructed for Strong Earthquake Observation Center at Beijing in 1981.

THEORETICAL STUDIES

Several researchers (Arya et al. 1978; Chandrasekaran 1970; Lin et al. 1986; Malushte et al. 1989; Mittal 1971; Mostaghel et al. 1983, 1983a; Newmark 1965; Qamaruddin 1978; Qamaruddin et al. 1986, 1986a; Younis et al. 1984; Westermo et al. 1983) have investigated behavior of sliding systems subject to harmonic and earthquake-type excitation. Mostaghel (1983) and Qamaruddin (1978) carried out research work on the behaviour of single-degree-of-freedom flexible systems on sliding base under different input excitations. To establish the effectiveness of the FSI scheme for masonry structures, the analytical studies made by the researchers are discussed in the following sections.

1. Mathematical Model

Qamaruddin (1978) and Arya et al. (1978) considered a single-story masonry house (Figure 3) to introduce the concept of FSI by providing sliding base in the structure. A thin layer of sliding material is interposed between the contact surfaces of bond beam of the superstructure and plinth band of the substructure. The structure is idealized as a two-mass system, with masses lumped at top and bottom. Horizontal wall stiffness and equivalent viscous damping are taken into account. Systems subjected to earthquake-induced (or harmonic) ground motions, is non-linear, due to the rigid-plastic behavior of the sliding support. But, the system becomes linear in each sliding and non-sliding phase. The conditions for the determination of transition point between any two phases may be defined. Whenever the inertia force of the two-mass system becomes equal to the frictional force (between the sliding surfaces), sliding begins. As long as the sliding velocity of the system is zero, the superstructure moves with the substructure. In this phase, the seismic analysis is exactly the same as for a conventional one degree-of-freedom system. The bottom mass of the FSI system either stops or continues to slide throughout the time history of earthquake ground motion. Numerical integration of the equations of motion has been made employing Runge-Kutta fourth order method.

2. Responses to Earthquake Excitations

The physical properties of masonry building were represented by a range of parameter values to arrive at generalized results (Qamaruddin 1978 and Qamaruddin et al. 1986, 1986a). The parameters are time period, mass ratio (m), dry frictional coefficient (F) and viscous damping. The seismic response was determined using 1940 El Centro earthquake and Koyna (India) 1967 earthquake. The quantities of interest for estimating realistic forces and displacements of the sliding structure are: (1) absolute acceleration which determines the forces acting on the shear walls; (2) the maximum relative displacement of the superstructure; and (3) the residual relative displacement which will indicate the position of the superstructure at the end of the ground motion.

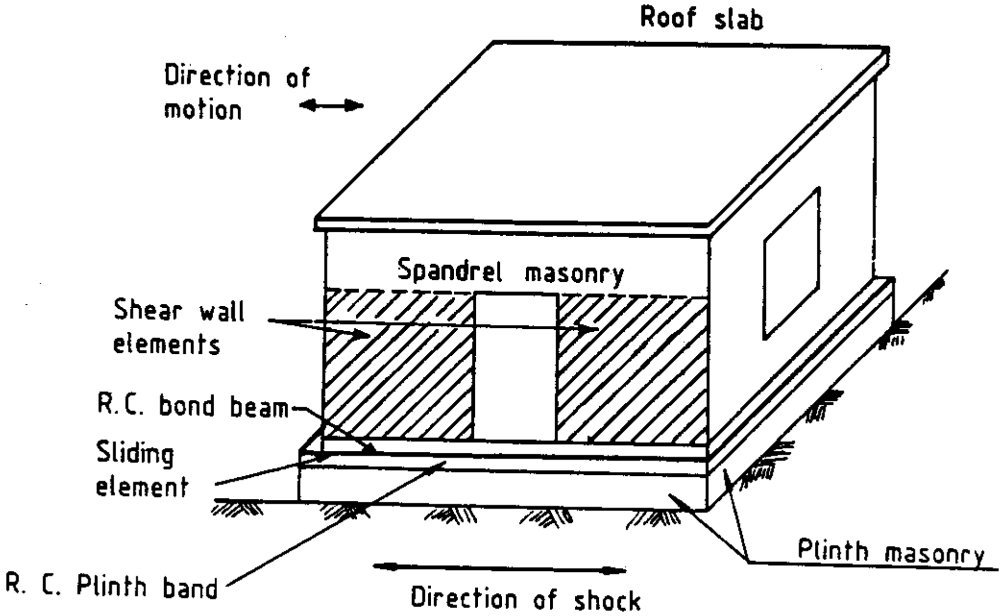

Fig. 3 Idealized friction seismic isolated brick house

The results of the seismic response computation are presented in the form of frictional response spectra. The absolute acceleration, the maximum relative displacement and also the residual or permanent relative displacement are plotted against undamped fundamental period of the masonry buildings for different parametric combinations. Representative results of the spectra developed (Qamaruddin et al. 1986a) is shown through Figure 4. It is observed that as the mass ratio increases, the spectral acceleration decreases for both the earthquakes. It is also seen from these figures that the spectral acceleration decreases as the coefficient of friction decreases. Unlike the fixed-base structure, in the case of a sliding base structure the frictional spectra are generally flat and the values do not change much as the period of the system as well as other parameters are varied. These figures also show that generally acceleration of the FSI system is much less than that of the corresponding fixed-base system subjected to earthquake ground motion. From this investigation, it also turns out that the residual displacement is slightly smaller than the maximum dynamic displacement, which may be used in the design of structure. The value of this displacement is less than 18 mm which occurs at F = 0.15 and this value is considerably less than the usual plinth projection.

Mostaghel et al. (1983, 1983a) presented a similar mathematical model as first proposed by Qamaruddin (1978) and Arya et al. (1978), a few years later for solving problem of single-degree-of-freedom structures supported on sliding substructure and subjected to harmonic and also earthquake support motions. The single story structures were subjected to 1940 El Centro and 1949 Olympia earthquake ground motions (Mostaghel et al. 1983a). Spectra for absolute accelerations, relative displacements, sliding displacements and residual sliding displacements were evaluated for three mass ratios, four coefficients of friction and a damping of 5 per cent critical. It is observed from the results of this investigation that for structures with time period less than 1.8 seconds the maximum sliding and residual sliding displacements are of the order of 1.25 times the peak ground displacements. It is also noted that increase in the levels of the input excitations increases the FSI system effectiveness in cutting down the acceleration. However, no attempt was made by Mostaghel et al. (1983a) to compare the analytical results with that of the experimental studies.

Su et al. (1989) carried out a comparative study for effectiveness of various base isolators. Seismic responses of a rigid structure with various base isolators subject to accelerograms of El Centro 1940 earthquake and Mexico City 1985 earthquake are evaluated. Based on the presented results, it is concluded that for relatively rigid structures, the FSI system is less sensitive to the undesirable variations in the frequency contents of ground excitation. Furthermore, due to their high-energy dissipation capacity, the FSI system can effectively reduce the transmitted acceleration with limited sliding displacements.

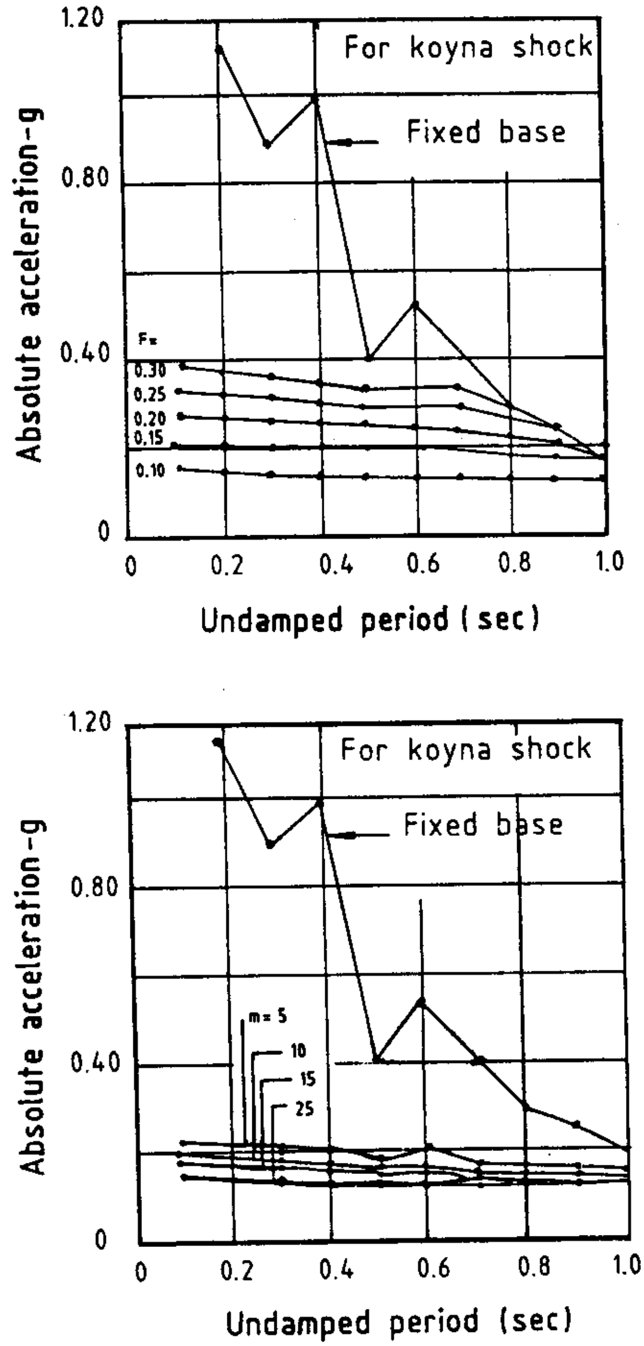

Fig. 4 Acceleration frictional response spectra

Tehrani and Hasani (1996) presented the seismic behavior of conventional real structures in Iran. Further on, they have performed experimental studies to select suitable materials, which permit sliding between the superstructure and the substructure of the sliding building system. The results showed that dune sand and lightweight expanded clay aggregates are good material for creating sliding layer in adobe building in Iran. The friction coefficient of sliding materials should be selected according to construction details to minimize the cost of building construction.

Tehrani and Hasani (1996) have considered the mathematical model for low-rise masonry buildings as a rigid body system for earthquake behavior and response study. The rigid structures were subjected to the three recorded Iranian earthquakes of high peak ground accelerations and El Centro earthquake of 18 May 1940. They obtained the well-established result as the friction coefficient (0.10 to 0.30) of the sliding layer is increased, maximum acceleration of the rigid structure is increased linearly up to a maximum value of base acceleration. But the relative velocity and displacement of the structure is decreased with the increase of the coefficient of friction.

Considering the vertical component of ground acceleration in addition to the horizontal one, Tehrani and Hasani (1996) have determined the horizontal acceleration of the sliding structure (using friction coefficient as 0.10) subjected to sixteen Iranian earthquake ground excitations. The results indicate that the increase in the computed horizontal acceleration of the sliding structure subjected to most of the earthquakes is less than 10% whereas this increase is over 50% in the case of Naghan, Tabas and Ab-Bar earthquakes. The maximum relative displacements in the sliding structure subjected to most of the earthquakes is less than 10 mm, but when the structure is subject to Vandik, Naghan, Deyhok, Tabas and Ab-Bar earthquakes, this response increases up to 13 mm.

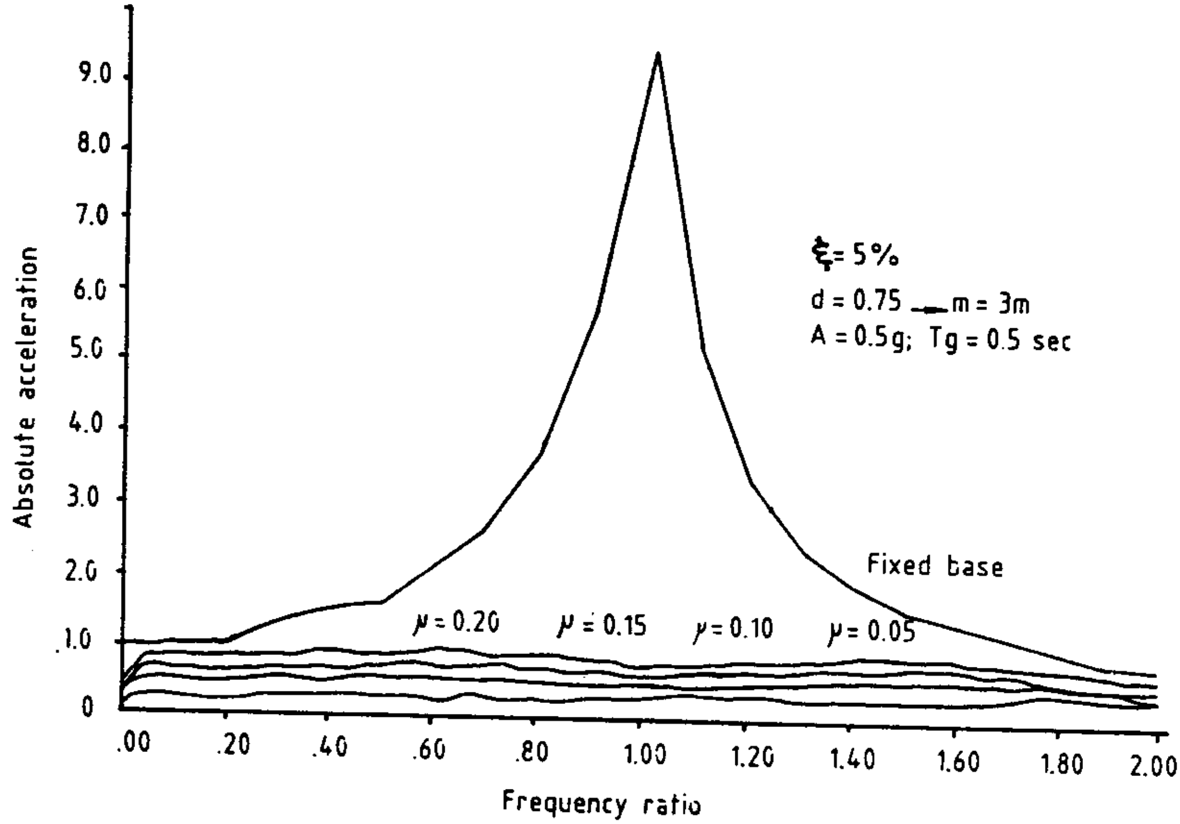

Fig. 5 Variation of acceleration with frequency ratio

3. Responses to Dynamic Excitations

Representative Figure 5 shows the normalized acceleration spectrum for sliding structures subjected to harmonic support motion (Mostaghel et al. 1983). It is seen from Figure 5 that the spectral response of the isolated structures appears to be independent of frequency whereas the level of the response depends only on the coefficient of friction. The response level of the sliding structures is considerably less than that of the corresponding fixed-base structures. The sliding displacement is larger for smaller coefficients of friction while it does not vary significantly with frequency ratio (Mostaghel et al. 1983).

Zongjin et al. (1989) determined the response of the sliding system subjected to the forced vibration by employing the same mathematical model as introduced earlier by Qamaruddin (1978) and Arya et al. (1978) and used by other researchers (Arya 1984 and Qamaruddin et al. 1986, 1986a). The analytical results showed that the absolute acceleration of the superstructure of the sliding structure was significantly reduced from the fixed-base condition. The results also demonstrated that the response of the sliding structure was nearly independent of intensity of the input motion.

4. Response of Pure-Friction Sliding Structures to Bi-Directional Harmonic Ground Motion

There have been only a few studies related to dynamic behavior of pure-friction sliding structures under bi-directional ground motion. Jangid (1997) investigated the response of structures with sliding support to two horizontal components of the harmonic ground motion. The mathematical model of an idealized one-story structure with a sliding support between the base mass and the foundation is shown in Figure 6. The system has four degrees-of-freedom which are the relative displacements of the superstructure (x_s and y_s) and the relative displacements of the base mass (x_b and y_b) in two orthogonal x- and y-direction, respectively. Jangid (1997) has employed Newmark's method for the solution of the governing differential equations of motion assuming linear variation of acceleration over a small time interval. The response of the system is expressed in the normalized form (normalized response = ratio of the peak response considering interaction of the frictional forces to the corresponding peak response without interaction of the frictional forces).

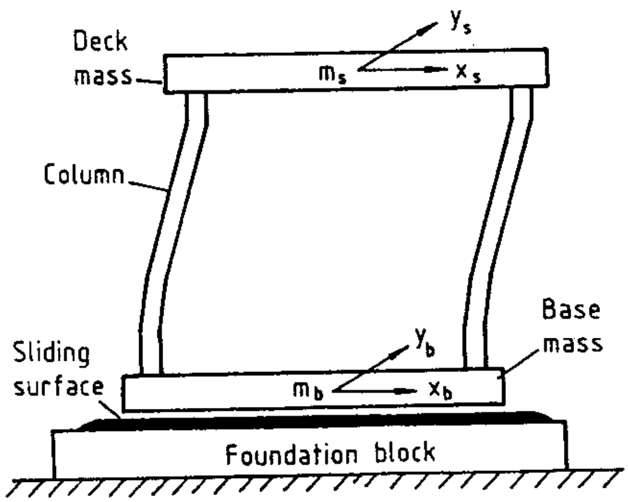

Fig. 6 Mathematical model with sliding support

To reduce the total number of parameters, the time period of the superstructure with a fixed-base is kept constant in the two orthogonal directions (i.e., $T_s = 2\pi/\omega_s$; ω_s is the fixed-base frequency of the superstructure). The frequency of the harmonic excitation in two orthogonal directions is the same, i.e., $\Omega = \Omega_x = \Omega_y$. The effects of bi-directional interaction of frictional forces are studied against the parameters Ω/ω_s, T_s, the phase difference of the ground motions (ϕ), the ratio of the ground motion amplitudes A_x and A_y in the x- and y-direction, respectively, the mass ratio and the friction coefficient (μ).

Figure 7 shows that the normalized absolute acceleration of the superstructure is less than unity for all frequencies of excitations. This indicates that the response with interaction of two component excitations is reduced in comparison to those without interaction. The normalized sliding base displacements are greater than unity which indicates that the sliding displacement under bi-directional ground motion considering the effect of interaction of frictional forces is higher in comparison to the same, but without interaction. In view of this, if the interaction of the frictional forces at the sliding support is ignored then the superstructure acceleration is overestimated and the sliding displacement is underestimated.

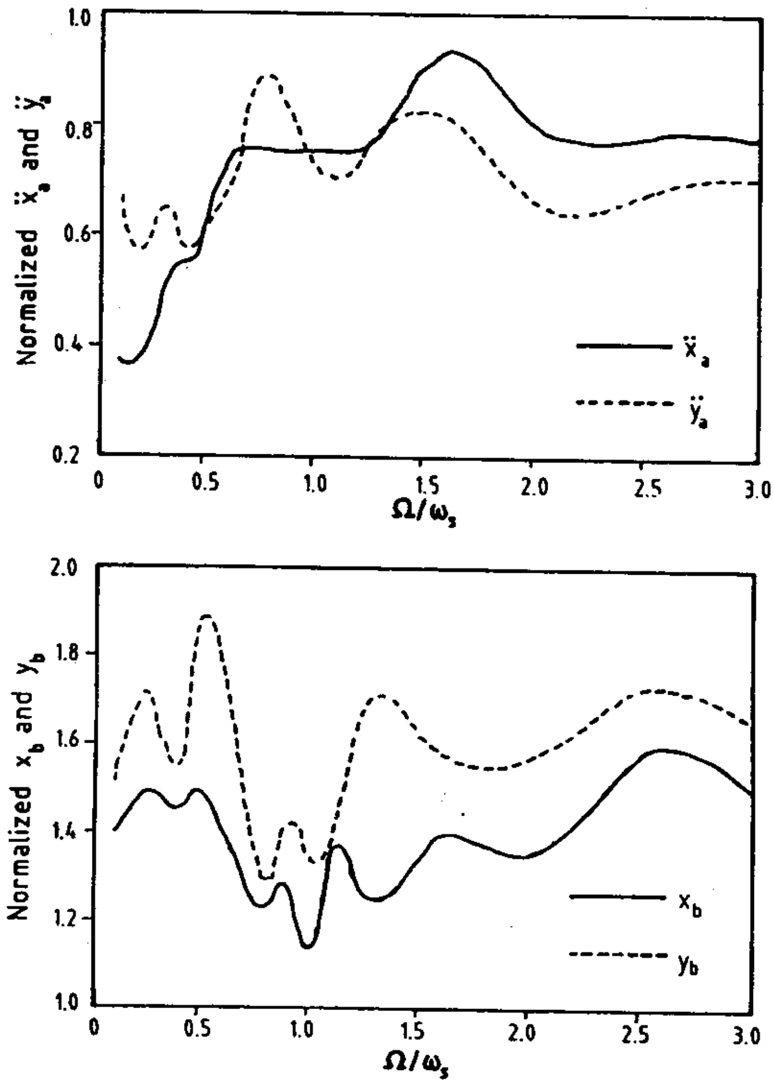

Fig. 7 Normalized superstructure acceleration and base displacement

Table 1: Normalized Absolute Acceleration and Base Displacement (T_s =0.5 s, m_b/m_s = 1)

Earthquake ground motion	μ	Normalized response					
		x_a	y_a	x_s	y_s	x_b	y_b
El Centro, 1940	0.01	0.673	0.730	0.673	0.729	1.173	1.094
	0.05	0.950	0.749	0.949	0.749	1.556	2.649
	0.10	0.828	0.771	0.826	0.772	1.110	1.181
	0.15	0.833	0.771	0.834	0.771	1.211	1.416
Olympia, 1949	0.01	0.887	0.811	0.887	0.811	1.064	1.300
	0.05	0.998	0.861	0.998	0.860	1.124	1.086
	0.10	0.907	0.912	0.907	0.911	1.237	1.000
	0.15	0.912	0.967	0.912	0.967	1.378	0.943
Taft, 1952	0.01	0.860	0.888	0.861	0.887	1.074	1.300
	0.05	0.919	0.900	0.919	0.899	1.418	1.484
	0.10	0.929	0.910	0.929	0.909	1.340	1.260
	0.15	0.971	0.831	0.971	0.830	1.838	0.887

The normalized responses of the sliding structures are investigated (Jangid 1997) for the horizontal components of El Centro 1940, Olympia 1949 and Taft 1952 earthquake ground motions. Table 1 shows the results for the normalized absolute acceleration of the superstructure and the base displacement for different friction coefficients of the sliding support for these three earthquakes ground motions. The absolute acceleration of the superstructure is relatively less for considering the interaction of the frictional forces as compared to those without interaction. The normalized response for relative displacement is almost the same as that of absolute acceleration in the two directions. But, the base displacement is higher for the two component earthquake excitation.

It is seen from Table 1 that the difference in the base displacement between the two cases is quite significant (refer to El Centro, 1940, $\mu = 0.05$). Further, the normalized base displacement in the y-direction is less than unity for $\mu = 0.15$ in the case of 1952 Taft earthqauke. This indicates that the base displacement in the y-direction is less when considering the effects of the interaction, in comparison to that without interaction. Thus, the superstructure accelerations are overestimated and sliding base displacements are underestimated if the effect of the bi-directional interaction of the frictional forces is ignored. The error in the response ignoring the interaction effects is quite significant and such effects should be considered for the effective design of sliding structures.

5. Response of Multistory Masonry Buildings

Buildings with FSI Scheme: The response of multistory masonry buildings with inter-floor discontinuity at different story-levels and at the base was studied (Qamaruddin et al. 1986b) by representing the system with two rigid blocks, each block replacing several stories clubbed together. A layer of sliding material was interposed between the contact surface of bond beams of the first and second blocks of the superstructure and between the bond beam of the superstructure and plinth band of the sliding substructure. The seismic response was computed for 1940 El Centro earthquake and 1967 Koyna shock. It is seen that the spectral acceleration of the sliding system is constant and independent of the frequency. It is also observed less that the spectral acceleration of the sliding system is less than that of the corresponding fixed- base system subjected to Koyna shock for all parameter combinations. Similar trend is generally found in the case of sliding structure subjected to El Centro shock.

Qamaruddin et al. (1990) introduced a mathematical model for multistory masonry buildings with FSI scheme. The structure is idealized as a three-mass-spring-dashpot model with each mass having one degree of freedom. The system was subjected to 1940 El Centro shock and 1967 Koyna earthquake. A wide range of parameter values was chosen so that the results of the seismic response could be generalized. The spectral acceleration was determined for various values of stiffness ratio, mass ratio, coefficient of friction and system damping ratio. The results showed that the absolute spectral acceleration of the system does not vary much for different parameter combinations. It is also observed that the response level of the sliding structures is lower than that of similar fixed-base structures.

A mathematical model for dynamic analysis of multistory masonry building with sliding-rocking system considering it as a multiple-degree-of-freedom in shearing mode was proposed by Bingze et al. (1990). The sliding system was subjected to the 1940 El Centro, Luan-he 1976 (China) and Tianjin 1976 (China) earthquake shocks. It is concluded from this study that the efficiency of FSI system increase as the peak ground acceleration increases and/or the coefficient of friction decreases. Dynamic response of the isolated system, considering the coupled horizontal and vertical ground motion, was also studied (Bingze et al. 1990). The results of the investigation showed that the vertical component of the earthquake ground motion is not much sensitive to sliding motion of the structures.

Lou et al. (1990) determined the seismic response of multistory brick buildings with FSI scheme employing a multiple-degree-of-freedom shear model ignoring the effect of floor rotational inertia. The results of the analysis showed that the seismic response was significantly limited by using graphite powder or screened gravel as sliding materials at the base of the multistory brick building.

The response of multistory masonry buildings with FSI scheme subjected to Koyna and El Centro earthquakes ground motions is computed through a multi-mass-spring-dashpot mathematical model (Qamaruddin et al. 1994). The parameters involved in the seismic analysis are coefficient of friction, number of stories of the building and type of earthquakes. The results are presented through graphs and tables for different parameter combinations. The results show the reduction in the response of the multistory buildings with FSI as compared with the corresponding conventional buildings.

Resilience Friction Slide Base Isolation (RFSBI) System: Zhou and Han (1996) have developed a resilience friction slide base isolation system for low-cost buildings. An optimum design for resilience friction slide base isolated building has been presented through an example of a demonstration building. A five-story brick building (Zhou and Han 1996) was located in an area of seismic intensity of VIII (based on seismic intensity scale of China, which is similar to the Modified Mercalli Intensity). Fan et al. (1994) published structural details of the RFSBI system. The demonstration building is 42 m long and 9 m wide with a uniform interstory height of 2.8 m. Friction sliding isolation bearings consist of 30 cm × 30 cm square teflon pads, whereas a restoring element consists of spring steel bars.

Multi-mass-spring shear mathematical model is considered for seismic response analysis of the building because the configuration of the building in plan and elevation is regular and symmetric. Relative spring rigidity, K_s (ratio of base spring constant K_1 to the first story spring constant K_2) and friction coefficient (F) are basic parameters of the RFSBI system. Zhou and Han (1996) have carried out the time history analyses for the RFSBI system subjected to El Centro earthquake (N-S component) of 18 May 1940 with peak accelerations scaled to 0.212g, 0.425g and 0.850g. Equivalent base shear force and sliding drift of the friction slide bearings are adopted as evaluating criteria for working quality of the RFSBI system and seismic safety of the structure. The influence of F and K_s on the distribution of inter-story shear force along the building height is shown in Figure 8 for F = 0.13 and various value of K_s. The system is subjected to 1940 El Centro earthquake with 0.425g peak acceleration. Figure 8 shows that the influence of K_s on the shear force distribution is predominant on the lower story of the structure.

Several sets of F and K_s values have been chosen to perform the non-linear dynamic response analysis for the building to get the optimum design for RFSBI system. For the design earthquake with seismic intensity of VII, VIII, IX and X, the respective adopted values of peak ground acceleration are 0.105g, 0.212g, 0.425g and 0.85g. The results of the seismic response analysis show that the real value of the friction coefficient is usually greater than the designed one. For example the design value of F determined from the laboratory test for the sliding device in the building is 0.10 but the real value of F determined from the field test is 0.13. The results also indicate that the influence of the earthquake input upon the base sliding drift is more significant than the inter-story shear force induced in the superstructure of the building. Finally it turns out from this study that the uncertainty of the rigidity coefficient of the spring is smaller than that of F value.

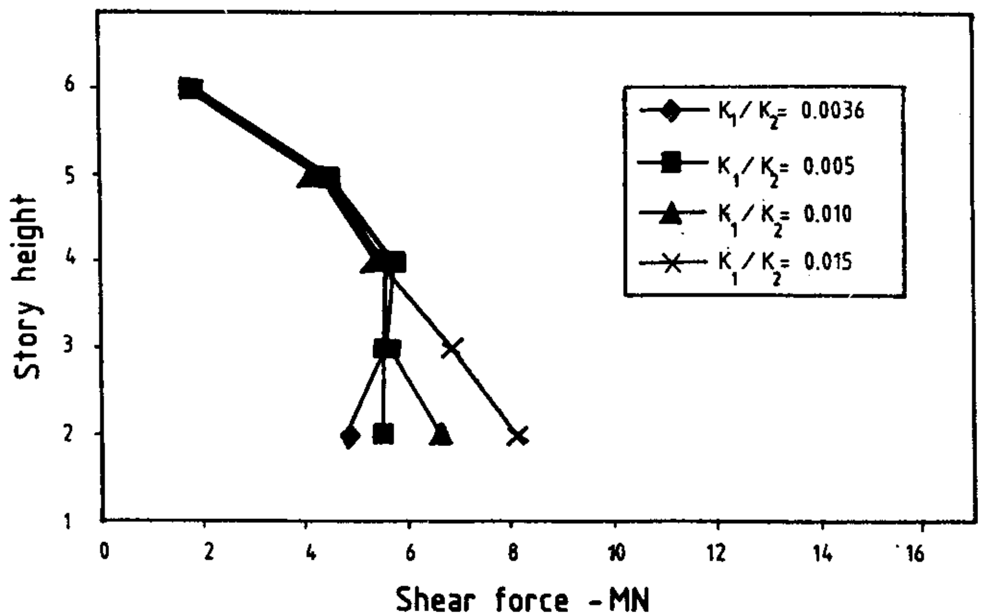

Fig. 8 Variation of inter-story shear force with base isolated building height

SUITABILITY OF FSI SYSTEM

Generally, both experimental evidence and analytical studies on the friction isolation system have demonstrated its beneficial seismic performance. Especially in the case of low- to medium-rise structures the FSI system successfully limits part of the seismic input energy transmitted to the structure. Major part of such energy is dissipated at the sliding base level of the structure. The structure with the FSI system performs almost independently of the dominant frequency content of earthquake excitation, and therefore such system is suitable for the structures to be located in near- or far-field earthquake environment and to be constructed on different soil conditions.

In the case of the FSI system, sliding displacements are larger compared to other systems. However, these displacements are within practical limit having an upper bound of about 1.25 times the maximum ground displacement. Structural displacements are limited to a level below the damage threshold of the system due to the mitigated acceleration response. This statement is generally valid for the frictional coefficient value of 0.15 for masonry buildings constructed in 1:6 cement-sand mortar. The coefficient of friction values could be higher for masonry buildings built with richer cement-sand mortar.

In view of the above-mentioned reasons, the FSI system has the potential of being an effective and simple alternative solution for protection against earthquakes, especially for masonry buildings and other rigid structures. However, there are several problems still to be solved in the application of the FSI system to real life structures. The optimal frictional coefficient range should be determined, because the maximum ground acceleration, which can be transmitted to the superstructure, is controlled by the frictional coefficient value of the sliding material. The upper bound of the frictional coefficient is supposed to be a value, which initiates sliding at (or below) the damage threshold of the structure. The lower bound of the frictional coefficient should be determined by analyzing the structure subjected to maximum wind force. The frictional resistance at the initiation of sliding should always be greater than the maximum wind force experienced by the structure. Also, low-intensity earthquake should not cause sliding of the structure. The choice of the sliding material should be considered with full attention taking into account the requirements of: durability, resistance of contamination from environmental conditions, good wear resistance and desirably low cost.

WORLDWIDE SEISMICALLY-ISOLATED MASONRY BUILDINGS

The UC Berkeley (Protective Systems Research Group (1997)) has developed a list of 197 worldwide seismically isolated buildings. Twelve seismically isolated masonry buildings selected from this list are shown in Table 2.

Table 2: Worldwide Seismically Isolated Masonry Buildings

No.	Building Name	Location	Super-structure	Isolation System	Remarks
1.	Apartments and Earthquake Observatory	Beijing, China	4-story brick masonry	Sliding base with sand layer	Built in 1981
2.	Equipment Building	Hashi, Xinjiang, China	3-story brick masonry	Sliding base with sand layer	Built in 1983
3.	Apartment Building	Xichang, Sichuan, China	6-story brick masonry	Graphite & lime mortar	Constructed in 1985
4.	Residential House	Montreal, Canada	3-story brick masonry	Friction dampers	Built in 1988
5.	Salt Lake City and County Building	Salt Lake City, Utah, USA	5-story brick and sandstone masonry	Lead and natural rubber bearing	Retrofit in 1989
6.	Apartment Building	Darli, Yunnan, China	6-story brick masonry	Slide steel piece	Built in 1992

Table 2 (Cont.): Worldwide Seismically Isolated Masonry Buildings

No.	Building Name	Location	Super-structure	Isolation System	Remarks
7.	Apartment Building	Xian, Xanshi, China	8-story brick masonry	Slide steel piece	Constructed in 1993
8.	Campell Hall	Monmouth,OR, USA	3-story masonry	Lead and natural rubber bearing	Retrofit in 1993
9.	Apartment Building and Shops	Xian, Xanshi, China	6-story brick masonry	Slide steel piece	Constructed in 1993
10.	Building No. 610, Stanford University	Palo Alto, California, USA	1-story brick masonry	Friction dampers	Built in 1915 & Retrofit in 1994
11.	Five Apartment Buildings	Xichang, Sichuan, China	6-story brick masonry	High damping rubber	Constructed in 1995
12.	Seven Garden House Buildings	New Delhi, India	N. A.	Friction dampers	Constructed in 1997

N. A.: Full information not available

There are only six masonry buildings with the FSI system. Cheaper sliding materials were used only in three such buildings. Slide steel piece was also tried in three masonry buildings as an alternative base isolation scheme. The FSI system has not gained much popularity, although this scheme has been used for seismic design of some actual buildings. This may be due to the fact that in reality, the key challenge that related to the FSI system is to develop the practical schemes for implementation in building construction. Also, building code provisions for the design of base isolated buildings are still in the developing stage and the total construction costs are likely to increase. In China, twelve base isolated masonry buildings have been constructed during 1981 to 1995. Generally, the number of stories in these buildings varies between three and eight. Eight six-story masonry base isolated buildings were constructed in China. High damping rubber was employed as an isolation system in six such buildings. Mainly, friction dampers have been used in masonry base isolated buildings in Canada, India and USA whereas in two cases, lead-rubber and natural rubber bearings were used as isolation system.

CONCLUSIONS

A state-of-the-art review of FSI scheme for masonry buildings is presented in this paper through the experimental and theoretical studies. It turns out from the study that the FSI system successfully limits part of the seismic input energy transmitted to the structure. Such a system performs almost independently of the dominant frequency content of earthquake excitation. Therefore, the FSI scheme is suitable for the structures located in near- or far-field seismic environment and founded on different soil conditions. The sliding displacements for the FSI system are large but are within practical limits. So, it may be suggested that the suitable frictional coefficient range for masonry structures would be between 0.10 to 0.20.

Although the concept of friction seismic isolation has been used for seismic design of some actual buildings, yet it has not gained wide popularity. Because, building code provisions for the design of base isolated buildings are still in the developing stage and the total construction costs are likely to increase. The FSI scheme is easy, economically feasible and requires higher degree of supervision for the construction of masonry structure. Therefore, it may be concluded that the FSI system is a viable alternative with respect to conventional system for improved seismic performance.

It is recommended that experimental research investigation should be carried to develop construction details of friction seismic isolation scheme for low-to medium-rise masonry buildings specifically to be constructed in developing countries. The cost-benefit investigation should also be carried out to develop practical schemes feasible for design applications in masonry buildings with friction seismic isolation system.

REFERENCES

1. Arya, A.S. (1967). "Design and Construction of Masonry Buildings in Seismic Areas", Bull. Indian Soc. of Earthquake Tech., Vol. 4, No. 2., pp. 25-37.
2. Arya, A.S., Chandra, B., and Rani, P. (1977). "Influence of Natural Disasters (earthquakes) on Educational Facilities, Part I - Collection and Analysis of Available Information - Annotated Bibliography", Report of School of Research and Training in Earthquake Engineering, University of Roorkee, India.
3. Arya, A.S., Chandra, B. and Qamaruddin, M. (1978). "A New Building System for Improved Earthquake Performance", Proc. of the Sixth Symposium on Earthquake Eng., University of Roorkee, India, Vol. I, pp. 499-504.
4. Arya, A.S. (1984). "Sliding Concept for Mitigation of Earthquake Disaster to Masonry Buildings", Proc. 8th World Conference on Earthquake Eng., San Francisco, Vol. 5, pp. 951-958.
5. Beck, J.L. and Skinner, R.I. (1974). "The Seismic Response of Reinforced Concrete Bridge Pier", J. Earthquake Eng. and Structural Dynamics, Vol. 2, pp. 343-358.
6. Bingze, S., Changrui, Y., Xiaolin, Z. and Siyuan T. (1990). "Experimental Study and Seismic Response Analysis of Multistory Brick Buildings with Friction Base Isolation", Proc. 5th North Am. Masonry Conference, Univ. of llinois, Urbana-Champaign, pp. 177-787.
7. Blakeley, R.W.G. et al. (1979). "Recommendations for the Design and Construction of Base Isolated Structures", Bull. New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2.
8. Buckle, I.G. and Mayes, R.L. (1990). "Seismic Isolation: History, Application and Performance - A Worldview", J. Earthquake Spectra, Vol. 6, No. 2, pp. 161-201.
9. Caspe, M.S. (1970). "Earthquake Isolation of Multistory Concrete Structures", J. Am. Con. Inst., Vol. 67, pp.923-933.
10. Chandrasekaran, A.R. (1970). "Earthquake Response of Friction Mounted Masses", Bull. Indian Soc. of Earthquake Tech., Vol. 7, No. 1, pp. 47-53.
11. Chopra, A.K., Clough, D.P. and Clough, R.W. (1973). "Earthquake Resistance of Building with a Soft First Story", J. Earthquake Eng. and Structural Dynamics, Vol. 1, pp. 347-355.
12. Earthquake resistant regulations (1992). "A World List - International Association of Earthquake Eng."
13. Fintel, M. and Khan, F.R. (1969). "Shock-Absorbing Soft-Story Concept for Multistory Earthquake Structures", J. Am. Con. Inst., Vol. 66, pp. 381-390.
14. Fan, S.Y. et al. (1994). "A Demonstration Building of Masonry Structure and its Earthquake Behavior Analysis", Proc. 3rd State Conference on Masonry Structure, Chendou, China.
15. Fu, Yu-An (1988). "Sliding-Vibrating Response of Elastic Structures", Bull. New Zealand National Soc. for Earthquake Eng., Vol. 21, pp. 190-197.
16. Gee, E.R. (1934). "Dhubri earthquake of 1930", Memoirs of Geological Survey of India, LXV, pp. 1-106.
17. Jangid, R.S. and Datta, T.K. (1995). "Seismic Behavior of Base-Isolated Buildings: a State-of- the-Art Review", Proc. Institution of Civil Engineers, Structures and Buildings, Vol. 110, pp. 186-203.
18. Jangid, R.S. (1997). "Response of Pure-Friction Sliding Structures to Bi-Directional Harmonic Ground Motion", J. Engineering Structures, Vol. 19, No. 2, pp. 97-104.
19. Kelly, J.M. (1984). "Aseismic Base Isolation: Review and Bibliography", Journal of Soil Dynamics and Earthquake Eng., Vol. 5, pp. 202-216.
20. Krishna, J. and Chandra, B. (1969). "Strengthening of Brick Buildings in Seismic Zones", Proc. 4th World Conference on Earthquake Eng., Chile.
21. Li, L. (1984). "Base Isolation Measure for Aseismic Buildings in China", Proc. 8th World Conference on Earthquake Eng., San Francisco, California, pp. 791-798.
22. Lin, B.C. and Tadjbakhsh, I.G. (1986). "Effect of Vertical Motion on Friction-Driven Isolation Systems", J. Earthquake Eng. and Structural Dynamics, Vol. 14, pp. 609-622.

23. Lou, Y., Wang, M. and Su, Z. (1992). "Research of Sliding Shock Absorbing of Multistory Brick Buildings", Proc. 10th World Conference on Earthquake Eng., Madrid, Spain, pp. 2499-2504.

24. Malushte, S.R. and Singh, M.P. (1989). "A Study of Seismic Response Characteristics of Structures with Friction Damping", J. Earthquake Eng. and Structural Dynamics, Vol. 18, pp. 767-785.

25. Megget, L.M. (1978). "Analysis and "Design of a Base-Isolated Reinforced Concrete Frame Building", Bull. New Zealand National Soc. for Earthquake Eng., Vol. 11, No. 4.

26. Mittal, K.C. (1971). "Sliding and Overturning of Objects during Earthquakes", M. E. Thesis, University of Roorkee.

27. Moinfar, A.A. (1972). "Earthquake Resistant Design of Brick Masonry Buildings", European Symposium on Earthquake Eng., London, England.

28. Monge, J.E. (1969). "Seismic Behavior and Design of Small Buildings in Chile", Proc. 4th World Conference on Earthquake Eng., Chile.

29. Mostaghel, N., Hejazi, M. and Tanbakuchi, J. (1983). "Response of Sliding Structures to Harmonic Motion", J. Earthquake Eng. and Structural Dynamics, Vol. 11, pp. 355-366.

30. Mostaghel, N. and Tanbakuchi, J. (1983a). "Response of Sliding Structures to Earthquake Support Motion", J. Earthquake Eng. and Structural Dynamics, Vol. 11, 729-748.

31. Newmark, N.M. (1965). "Effect of Earthquakes on Dams and Embankments", Geotechnique, The Ins. of Civil Engineers, Vol. 15, pp. 139-160.

32. Nikolic-Brzev, S. and Arya, A.S. (1990). "Seismic Isolation Systems for Multi-Story Masonry Buildings", Proc. National Seminar on Earthquake-Resistant Design of Structures, New Delhi, pp. I-5.1 to I-5.20.

33. Nikolic-Brzev, S. (1994). "An Innovative Seismic Protection Scheme for Masonry Buildings", Proc. 10th International Conference on Brick/Block Masonry, Calgary, Canada,Vol. 1, pp. 273-282.

34. Nikolic-Brzev, S. and Arya, A.S. (1996). "Seismic Isolation of Masonry Buildings – An Experimental Study", Proc. 11th World Conference on Earthquake Eng., Paper No. 1338, Acapulco, Mexico.

35. Plummer, H.C. and Blume, J.A. (1953). "Reinforced Brick Masonry – Lateral Force Design", Structural Clay Products Institute, Washington.

36. Paulson, T.J., Abrams, D.P. and Meyes, R.L. (1991). "Shaking-Table Study of Base Isolation for Masonry Buildings", J. Structural Engineering, ASCE, Vol. 117, pp. 3315-3336.

37. Qamaruddin, M. (1978). "Development of Brick Building Systems for Improved Earthquake Performance", Ph.D. Thesis, September 1978, University of Roorkee, Roorkee.

38. Qamaruddin, M., Arya, A.S. and Chandra, B. (1978). "Experimental Evaluation of Seismic Strengthening Methods of Brick Buildings", Proc. Sixth Symposium on Earthquake Eng., University of Roorkee, Vol. I, pp. 353-359.

39. Qamaruddin, M., Chandra, B. and Arya, A.S. (1984). "Dynamic Testing of Brick Building Models", Proc. of the Institution of Civil Engineers (London), Vol. 77, Part 2, pp. 353-365.

40. Qamaruddin, M., Arya, A.S. and Chandra, B. (1986). "Seismic Response of Brick Buildings with Sliding Substructure", J. Structural Engineering, ASCE, Vol. 122, No. 3, pp. 558-572.

41. Qamaruddin, M., Rasheeduzzafar, Arya, A.S. and Chandra, B. (1986a). "Seismic Response of Masonry Buildings with Sliding Substructure", J. Structural Engineering, ASCE, pp. 122, No. 9, pp. 2001-2011.

42. Qamaruddin, M., Ali, S.M. and Qadeer, A. (1986b). "Seismic Response of Multistoried Masonry Buildings with Inter-Floor Discontinuity", Proc. Eight Symposium on Earthquake Eng., Roorkee, Vol. I, pp. 327-334.

43. Qamaruddin, M., Qadeer, A. and Ali, S.M. (1990). "Response of Masonry Building with Seismic Base Isolation", Proc. Fifth North Am. Masonry Conference, Univ. of Illinois, Urbana-Champaign, pp. 189-200.

44. Qamaruddin, M., Al-Jabri, K.S. and Mauroof, A.L.M. (1994). "Seismic Response of Multistory Masonry Buildings with Friction Isolation System", Proc. Tenth Symposium on Earthquake Eng., Roorkee, pp. 303-314.

45. Skinner, R.I., Beck, J.L. and Bycroft, G.N. (1975). "A Practical System for Isolating Structures from Earthquake Attack", J. Earthquake Eng. and Structural Dynamics, Vol. 3, pp. 297-309.

46. Skinner, R.I. and McVerry, G.H. (1975). "Base Isolation for Increased Earthquake Resistance of Buildings", Bull. New Zealand National Soc. for Earthquake Eng., Vol. 8, No. 2.

47. Skinner, R.I. et al. (1980). "Hysteretic Dampers for the Protection of Structures from Earthquakes", Bull. New Zealand National Soc. for Earthquake Eng., Vol. 13, No. 1.

48. Steibrugge, K.V. and Flores, R. (1963). "The Chilean Earthquake of May 1960 – a Structural Engineering View Point", Bull. of the Seis. Soc. Am., Vol. 53, No. 2.

49. Su, L., Ahmadi, G. and Tadjbakhsh, I.G. (1989). "Comparative Study of Base Isolation Systems", J. Engineering Mech., ASCE, Vol. 115, No. 9, pp. 1976-1992..

50. Tehrani, F.M. and Hasani, A. (1996). "Behavior of Iranian Low Rise Buildings on Sliding Base to Earthquake Excitation", Proc. 11th World Conference on Earthquake Eng., Mexico City, Mexico, Paper No. 1433.

51. University of California at Berkeley (1997). "Protective System of Research Group of the Earthquake Research Center", UC Berkeley, USA.

52. Westermo, B. and Udwadia, F. (1983). "Periodic Response of a Sliding Oscillator System to Harmonic Excitation", J. Earthquake Eng. and Structural Dynamics, Vol. 11, pp. 353-366.

53. Yorkdale, A.H. (1970). "Masonry Building Systems", State of Art Report No. 9, Technical Committee No. 3, International Conference on Planning and Design of Tall Buildings, Lehigh University, USA.

54. Younis, C.J. and Tadjbakhsh, I.G. (1984). "Response of Sliding Rigid Structure to Base Excitation", J. Engineering Mechanics, ASCE, Vol. 110, pp. 417-432.

55. Zhou, X. and Han, M. (1996). "Optimum Design of Resilience-Friction-Slide Base Isolation System for Low Cost Buildings", Proc. 11th World Conference on Earthquake Eng., Mexico City, Mexico, Paper No. 269.

56. Zongjin, L., Rossow, E.C. and Shah, S.P. (1989). "Sinusoidal Forced Vibration of Sliding Masonry System", J. Structural Engineering, ASCE, Vol. 115, No. 7, pp. 1741-1755.