EARTHQUAKE HAZARD MITIGATION FOR RURAL DWELLINGS BY P-F BASE ISOLATION

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ABSTRACT

The concept of seismic isolation is applied here to rural dwellings by separating the super structure from the foundation at plinth level by a sliding earthquake energy reducing friction layer in the form of Green marble-Green marble sliding couple. The dynamic interface property of this sliding interface has been investigated. Shake table tests is performed on ½ scaled single story base isolated brick masonry model supported by sliding interface subjected to an artificial accelerogram that is compatible with design spectrum of Indian standard (IS 1893 (Part 1): 2002) corresponding to the level of maximum considered earthquake in the most severe seismic zone (PGA=0.36g) in horizontal and 2/3rd of this acceleration in vertical direction. It is observed that the structure with green marble interface is quite effective in isolating the structure from support excitation. 70% reduction in absolute response acceleration at roof level for the isolated structure is obtained experimentally as compared to analytical fixed base structure at the cost of 50mm peak sliding displacement which is well within plinth projection of 75mm (3in) and can be used as a low cost pure friction (P-F) base isolation for rural masonry dwellings.

KEYWORDS: earthquake hazard mitigation, rural dwellings, pure friction base isolation.

1. INTRODUCTION

Masonry construction is the most widely used form of rural housing in developing countries like China, India, Iran, Turkey, etc. There are several advantages of masonry construction vis-à-vis the alternatives in the form of reinforced concrete and steel, namely, thermal insulation, acoustics, ease of remodeling after construction, easy and inexpensive repair, use of locally available materials, need of less skilled labour, less engineering intervention, etc. However, poor seismic withstand capacity is a major hindrance for its use in seismically active regions. To improve the seismic behaviour of masonry constructions, the seismic base isolation by pure friction (P-F) sliding system has been explored wherein a sliding layer is introduced at plinth level, on which the super structure simply rests and is free to slide except for friction resistance. The smaller the coefficient of friction the lesser is response acceleration and base shear as the super-structure slides across the plinth beam during a strong shaking. There is no re-centering mechanism to bring the super-structure back to its original position after the earthquake subsides and some residual sliding displacements remain. Special attention needs to be given to keep these displacements within acceptable limits. Different sliding interface materials have been examined in past studies (see, e.g., Arya, et al. 1978, Qummarudin, et al. 1986, Li 1984, Bingze et al. 1990, Nikolic-Brzev and Arya 1996, Lou et al. 1992, Tehrani and Hasani 1996). In general, a lower coefficient of friction leads to larger sliding displacement and a usable range of friction coefficient has been postulated to be in the range (0.05–0.15). Previous investigations have revealed that Teflon (PTFE) sliding against stainless steel has a low coefficient of friction value in the most desirable range, i.e., 0.05 to 0.15. However, a continuous bonding of stainless steel sheet over concrete course is very expensive and leads to construction difficulties. Graphite, grease, screened sand, dry and wet sand are economical alternative but they can not be used for a long term as grease can be contaminated by debris, dirt, etc.; graphite can be affected by chemical reaction; and sand may get crushed after the shock which will then increase the frictional resistance. Thus there is a need for search of alternative interface materials which may be easily available, economically viable and which can be readily adopted in construction. The present investigation is to explore alternative low cost P-F sliding interface materials in the form of Green marble/Green marble for seismic protection of rural dwellings through an analytical and experimental study.

2. MATHEMATICAL IDEALIZATION

A building unit with P-F sliding joint is idealized as two degree of freedom system as shown in Figure 1. The structure above the sliding joint is assumed to remain elastic and the mass of the roof in addition to one half the mass of the wall is lumped at the roof (M_t) while the rest is lumped at the base with the mass of the bond beam (M_b). The base mass is assumed to rest on a plane with dry friction damping to permit sliding of the system. A model incorporating static and Coulomb friction is used for frictional force at sliding interface for stick-slip non-linear behavior of sliding joint.

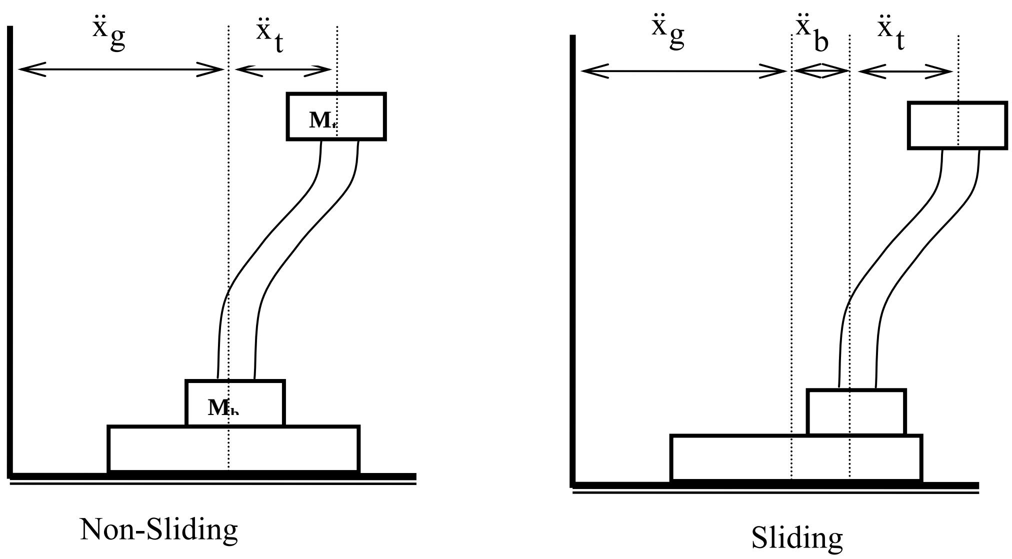


Figure 1: Schematic diagram for non-sliding and sliding system

Let $\ddot{x}_g$ denote the ground acceleration; x_t and x_b represent the relative displacement of top mass with respect to bottom mass and relative displacement of the bottom mass with respect to ground respectively; and θ $(= M_t / M_b)$ be the mass ratio (MR). The natural frequency of the non-sliding system, ω_n is related to the stiffness (K) and the top mass as $\omega_n = \sqrt{\frac{K}{M_t}}$, $\xi(= C/2\omega_n M_t)$ is the fraction of critical damping. Further, let μ_s, μ_d, and μ_v denote the coefficients of static, kinetic/dynamic and viscous friction, respectively. The viscous coefficient of friction μ_v=0 for dry friction interface.

2.1. Non-sliding condition

The governing differential equation for non sliding condition can be obtained from equilibrium considerations as:

$$M_t(\ddot{x}_g + \ddot{x}_t) + C\dot{x}_t + Kx_t = 0$$
$$\text{or}, \ddot{x}_t + 2\xi\omega_n\dot{x}_t + \omega_n{}^2 x_t = -\ddot{x}_g$$

2.1. Sliding condition

Governing differential equation of motion of top mass and bottom mass for sliding case can be derived from equilibrium considerations:

$$\begin{aligned} &M_t(\ddot{x}_g + \ddot{x}_b + \ddot{x}_t) + C\dot{x}_t + Kx_t = 0 \\ &\text{or}, \ddot{x}_t + \ddot{x}_b + \omega_n\dot{x}_t + \omega_n{}^2 x_t = -\ddot{x}_g \end{aligned} \tag{2.1.1}$$

$$\text{and}, M_b(\ddot{x}_g + \ddot{x}_b) - C\dot{x}_t - Kx_t + F\,\text{sgn}(\dot{x}_t) = 0 \tag{2.1.2}$$

where, $\text{sgn}(x) = \begin{cases} 1, & x > 0 \\ -1, & x < 0 \end{cases}$, $F = (f_s + f_c)(M_t + M_b)g$ with $f_s = \mu_s$ = coefficient of static friction and f_c = Coulomb plus viscous friction component which is given by, $f_c = \mu_d \text{sgn}(\dot{x}_t + \mu_v|\dot{x}_t|)$ as recommended by Tan *et al.* (2001).

Equation 2.1.2 can be reorganised as:

$$\ddot{x}_b - 2\xi\omega_n\theta\dot{x}_t - \omega_n{}^2\theta x_t + (f_s + f_c)(1+\theta)g.\text{sgn}(\dot{x}_t) = -\ddot{x}_g$$

The non-sliding condition is determined when the horizontal inertia force of bottom mass does not exceed the opposing friction force at plinth level, i.e.

$$\left|C\dot{x}_t + Kx_t - M_{b(}(\ddot{x}_g + \ddot{x}_b)\right| < \mu_s(M_t + M_b)g$$
$$\text{or}, \left|2\xi\omega_n\dot{x}_t + \omega_n{}^2 x_t - 1/\theta(\ddot{x}_g + \ddot{x}_b)\right| < \mu_s(1 + 1/\theta)g$$

As long as the dynamic lateral force does not exceed, the frictional resistance of the interface, the bottom mass moves along with the base and the system acts as a single degree of freedom system. As soon as the force acting at the base exceed, the frictional resistance, the bottom mass begins to slide and this sliding can cease at any instant of time when ever the non-slip condition holds. Hence, at any time instant response of the building can be obtained by solving the two simultaneous equations, i.e., the sliding case and the non-slip condition. These equations are solved by Runge-Kutta 4th order solver in MATLAB-SIMULINK environment.

3. EXPERIMENTAL STUDY

The dynamic characteristics of the pure-friction interface for seismic protection of masonry buildings is investigated via friction and shake table tests at the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, India.

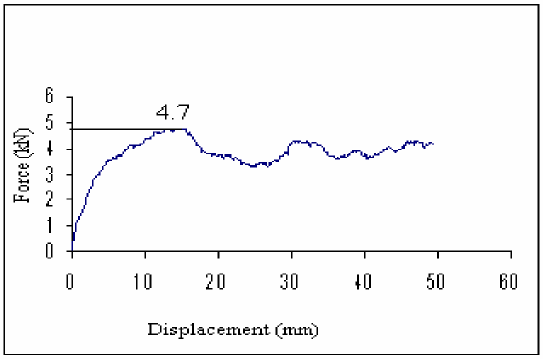
3.1. Friction test

The test setup (Figure 2) primarily consists of three units: shear box unit, normal load unit and shear load unit. The shear box unit is in circular shape, consists of two halves of 50mm depth each. It has been designed to accommodate samples of diameter 200mm. Upper half of the shear box is restrained against lateral movement whereas the lower half of the shear box is free to move laterally. The normal loading unit consists of a hydraulic jack and a reaction beam. The shear load unit consists of a servo-controlled actuator to apply the shear force or the horizontal load on the test specimens.

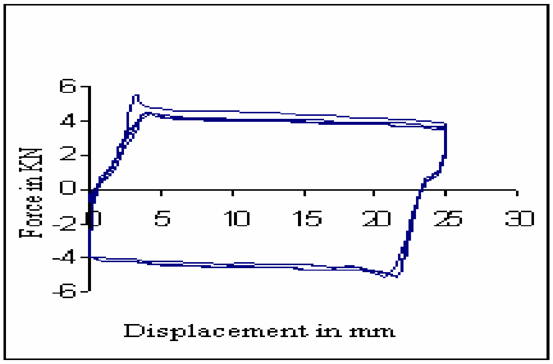
Figure 2: Experimental set up for friction test with shear box

Figure 3: Samples for friction test

Specimens of 200 mm diameter and 50 mm height (Figure 3) were prepared in 1:1.5:3 concrete cast with green marble in one side. Static tests were conducted under controlled displacement. The ramp rate had been kept constant at 0.5 mm/s with ramp limit of 25 mm and the normal load as 50 kN. Dynamic tests were carried out under displacement controlled conditions at frequency 0.25 Hz with ramp limit of 25 mm to give velocity of 12.5 mm/s. Dynamic tests were conducted on the same test set up with a small modification. Here the top box was restrained in both directions while bottom shear box attached to the actuator was allowed to move in both directions. The load displacement graphs for static and dynamic test are shown in Figure 4.



(a) Static test



(b) Dynamic test

Figure 4: The load displacement graphs

The coefficient of static friction from the static test is estimated as the ratio of the maximum shear force just before sliding and the normal load. It is observed that the force-displacement hysteretic loops of sliding models in dynamic test are rectangular in shape. The residual shear force can be approximately obtained from the force-displacement hysteretic graph. The friction coefficient can then be calculated by dividing the shear force by the normal force. From the test the coefficient of static friction and dynamic friction are obtained to be 0.09 and 0.08 respectively.

3.2. Shake table test

Shake table tests were performed on a half-scale single story brick masonry model with the proposed sliding interface. An artificial accelerogram that is compatible with design spectrum of Indian standard (IS 1893 (Part 1): 2002) corresponding to the level of maximum considered earthquake in the most severe seismic zone (with effective peak ground acceleration of 0.36 g) was used as the base excitation in horizontal direction. The vertical motion was characterized by the 0.67 times scaling of the horizontal motion.

A typical single room residential building is selected with one door opening in longitudinal wall, two windows in opposite cross walls and with an aspect ratio of 1:1 commonly used for traditional Indian housing practices. The layout plan of the building is shown in Figure 5.The model is constructed with specially manufactured half-scale bricks (114 mm x 57 mm x 38 mm) and 1:6 cement sand mortar. The model is constructed on a steel base plate, which is also used for mounting the specimen on the shake table platform. Reinforced concrete plinth beam is directly cast on this plate. A green marble strip of width same as that of plinth beam is pasted on the top of plinth beam with cement slurry. The smooth surface of the green marble on the plinth beam is kept exposed facing upwards. Another strip of green marble of width same as that of the bond beam is kept above the marble top on plinth beam with smooth surface down ward so that both smooth surfaces are in contact with each other. Two nos. 8 mm diameter Fe-415 steel with 6 mm links @150 mm spacing-bond beam reinforcement-is placed above this marble strip. Concreting is done so that the bond beam is directly casted on top of marble strip, and no leakage of concrete slurry to the interfaces takes place to avoid any untoward increase of friction between interfaces. As in the conventional construction practice, lintel bands are provided to ensure the integral box action of the building unit.

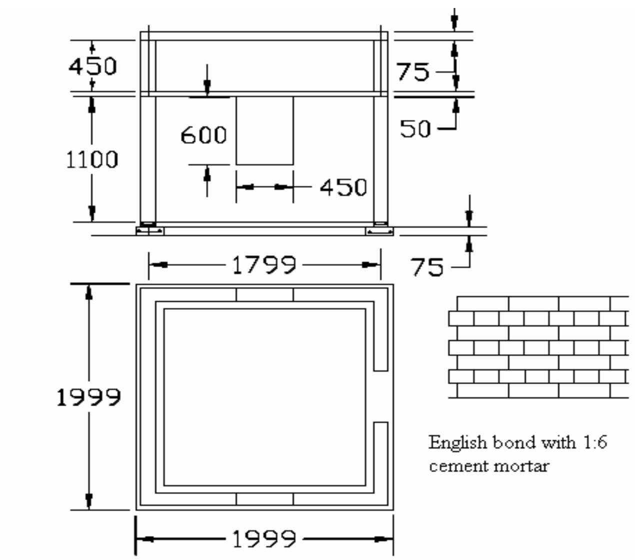


Figure 5: Plan and elevation (all dimensions in mm)

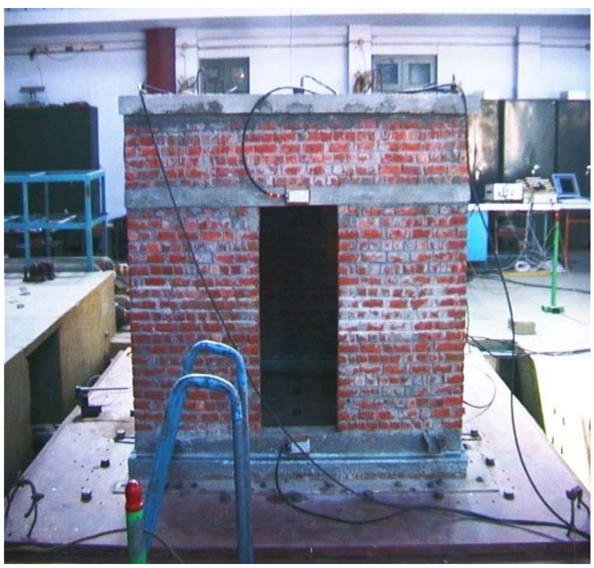


Figure 6: Building model and instrumentation

A digitally controlled shake-table facility capable of producing artificial earthquake compatible with a given response spectra for testing structure is used. Uni-axial force balanced accelerometers were mounted on the rooftop of the specimen and at the shake table platform to record the horizontal acceleration of the excitation and the response. An LVDT was also mounted on shake table platform for recording the relative displacement between plinth beam and bond beam. These accelerometers and LVDT were connected to the data acquisition system through a signal conditioner. Figure 6 shows the instrumentation of model. All accelerometer data have been passed through digital filters with pass band of 0.1-30 Hz which correspond to the frequency content of earthquake ground motion and also accounts for the baseline correction (Boore, 2005).

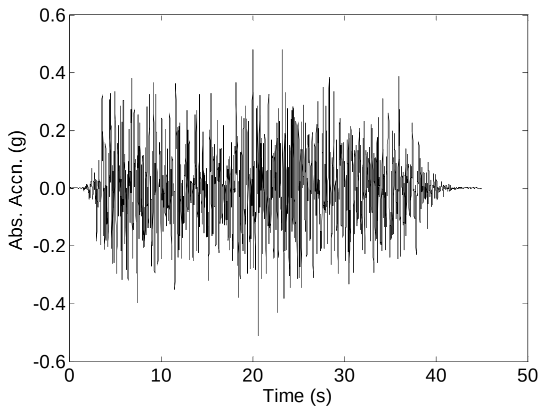
4. RESULTS AND DISCUSSION

Analytical predictions for absolute acceleration response of roof mass for sliding and fixed base un-cracked test structure subjected to the recorded table motion are compared with experimental results for validating the analytical model. Horizontal acceleration of the table platform is recorded through the sensor attached to the platform and is corrected for base line errors. The theoretical absolute acceleration response for fixed and sliding model is calculated for the recorded horizontal ground motion. The corrected platform motion has been considered as the horizontal input motion to analytical model. The natural frequency of the super-structure obtained from free vibration test is considered as 25.5 Hz in the analytical model. The mass ratio is taken as 2.2, which is consistent with the mass properties of the specimen. The damping ratio is assumed as 8% for the analytical model which is the average damping level for masonry (Booth and Key, 2006). The experimentally determined values of coefficients of friction for the sliding interface have been used in the analytical model. The horizontal accelerations at the roof level were recorded by the accelerometer mounted on the roof of the specimen.

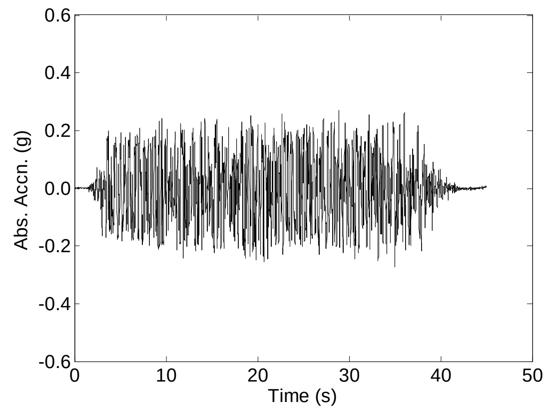
In the case of P-F isolation system the reduction in peak roof acceleration is achieved at the cost of increased sliding displacement. The relative displacement histories between foundation (plinth beam) and bond beam are recorded for the sliding interfaces through the LVDT attached to the shake table platform the direction of horizontal motion. Relative sliding displacement of the bottom mass has been calculated analytically for sliding and fixed base un-cracked model subjected to the recorded table motion. The corrected platform motion is considered as input motions in analytical model. Figure 7 represents ground motion and absolute acceleration response at roof level for sliding and fixed structure obtained analytically and experimentally. A comparison of the absolute acceleration amplification at roof level is shown in Table 1. Acceleration amplification ratio represents ratio between the response acceleration at roof level and the peak ground acceleration. While maximum roof acceleration increases by about 70% in the case of fixed base buildings, a reduction in maximum roof acceleration is observed in the sliding base models. A comparison in the reduction of response acceleration level for the isolated structure and fixed base structure can be obtained by comparing fixed base amplification ratio and sliding base amplification ratio. The reduction in response acceleration is (1.69-0.52)/1.69= 69% when compared with experimental response and (1.69-0.62)/1.69 = 63% when compared with the analytical response. The difference between experimental and analytical reduction in response acceleration is within 10%. These close agreements serve as validation of the analytical model with respect to maximum roof acceleration response. Analytical result for relative base sliding displacement is compared with experimentally observed displacement response and is shown in Figure 8. The maximum relative base sliding displacement is 50 mm (experimental) and 30 mm (analytical), which is well within the commonly provided plinth projection of 75 mm (3 in).

Table 4.1 Comparison of the absolute acceleration amplification at roof level

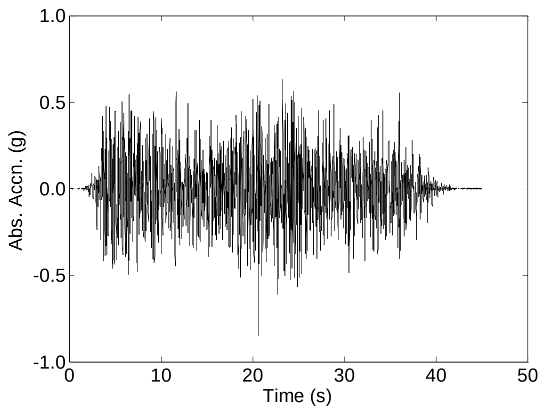
Maximum Absolute Table Acceleration (g)	Fixed		Sliding			
	Analytical		Analytical		Experimental	
	Acceleration (g)	Amp	Acceleration (g)	Amp	Acceleration (g)	Amp
0.51	0.86	1.69	0.32	0.62	0.27	0.52



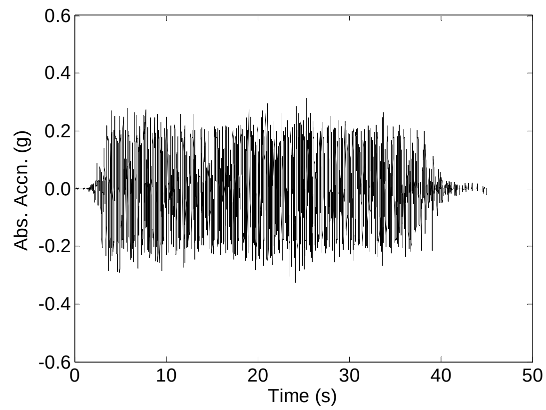
a) ground motion (Filtered)



b) Experimental roof acceleration responses (Filtered)



c) Analytical roof acceleration response (fixed model)



d) Analytical roof acceleration response (sliding model)

Figure 7 Ground motion and absolute acceleration response at roof level for sliding and fixed structure

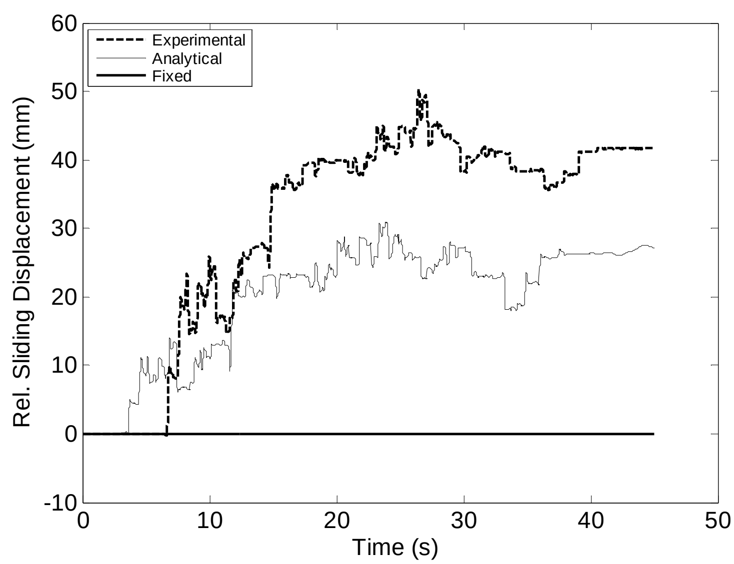


Figure 8 Comparative relative base sliding displacement responses.

5. CONCLUSIONS

The reduction in absolute response acceleration at roof level for the isolated structure with Green marble/Green marble interface obtained experimentally as compared to the fixed base structure is almost 70%. The difference in response reduction for analytical prediction and the experimental observation is within 10%. This reduction in absolute acceleration is observed at the cost of increased displacement at plinth level. It is observed that the peak relative sliding displacement 50 mm (experimental) and 30 mm (analytical) is well within plinth projection of 75 mm (3 in) and can be used as a low cost pure friction (P-F) base isolation in rural masonry dwellings for earthquake hazard mitigation. The cost of providing Green marble/Green marble interface works out to approximately 250 INR (6 USD) per meter run in India as against 800 INR (19 USD) per meter run for the case of Teflon/Stainless steel interface, which is commonly used for bridge bearings.

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