A NEW SYSTEM OF BRICK BUILDINGS FOR IMPROVED BEHAVIOUR DURING EARTHQUAKES

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ABSTRACT

The technique proposed for achieving earthquake resistance of masonry buildings consists of introducing a clear joint between the superstructure and foundation masonry at plinth level which would permit their relative sliding during a severe earthquake. Reinforced concrete bond beams are introduced just above and below the joint for integral action of respective components. Analytical seismic response calculations and experimental shake table tests on half size specimens definitely prove the reduction of effective acceleration acting on the superstructure masonry as compared with conventional buildings. The cracking damage is also seen to be much reduced and collapse avoided.

INTRODUCTION

The vulnerability of masonry buildings during earthquakes is well known. Some methods of strengthening such buildings have been developed (1,2,3,4) and included in some building codes (5,6). However, no effort had been made till recently for the seismic isolation of masonry buildings. The authors have tried a new technique for reducing the earthquake force on masonry buildings by providing a well defined sliding joint between superstructure of the building and its foundation at the plinth level. Observations during Dhubri Earthquake in 1930 and Bihar-Nepal Earthquake in 1934 in India had indicated that some buildings in which possibility of movements existed at their points of support suffered less damage than those buildings in which such freedom of movement was not available (7). The concept of the sliding joint centres around the idea that firstly the force transmitted to the superstructure will be limited to that required to cause sliding at the joint and, secondly, that energy will be dissipated during frictional sliding. The concept has been examined by theoretical dynamic response analysis as well as by half size specimens tested on shake table upto the ultimate strength (8). This paper presents typical analytical and experimental results obtained.

THEORETICAL STUDY

Mathematical Idealization of Sliding Type Building

The concept of the sliding joint at plinth level is shown in Fig. 1, through a single-storeyed brick building. It is assumed that a layer of suitable material with a known coefficient of friction is laid between the contact surfaces of bond beam of the superstructure and plinth band of the substructure. For computing earthquake response, the building is idealized as a two degrees of freedom discrete mass model as shown in Fig. 2. The spring action in the system is assumed to be provided by the shear wall, that

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is, those resisting shear forces parallel to the direction of earthquake shock. Internal damping is represented by a dashpot in parallel with the spring. The mass of the roof slab and that of half the height of walls is lumped at the roof level and half the mass of walls is lumped at the level of the bond beam. The two-mass system is then permitted to slide at the plinth level.

Equations of Motion

There will be three different phases in the complete motion-history of the sliding system due to the frictional resistance at its base (Fig. 2).

(a) Initially, so long as the acceleration of the moving system does not overcome the frictional resistance, mass M_b will move with the base since there will be no sliding, and the system would behave as a single degree of freedom system. Therefore the equation of motion would be:

$$\ddot{Z}_t + 2p\zeta(\dot{Z}_t - \dot{Z}_b) + p^2(Z_t - Z_b) = -\ddot{y}(t) \qquad \ldots (1)$$

in which each superscript dot represents a differentiation with time.

(b) The sliding of the bottom mass would begin when the frictional resistance at the plinth level is overcome by the force which causes sliding. The force to cause sliding S_F is given by

$$S_F = C_s(\dot{Z}_t - \dot{Z}_b) + K_s(Z_t - Z_b) - M_b\ddot{x}_b \qquad \ldots (2)$$

and sliding of bottom mass will occur if

$$|S_F| > \mu M_T g \qquad \ldots (3)$$

The system would now act as two degrees of freedom system for which the equations of motion can be written in a simplified form as follows:

$$\ddot{Z}_b - 2p\zeta\theta(\dot{Z}_t - \dot{Z}_b) - p^2\theta(Z_t - Z_b) + F = -\ddot{y}(t) \qquad \ldots (4)$$

$$\ddot{Z}_t + 2p\zeta(\dot{Z}_t - \dot{Z}_b) + p^2(Z_t - Z_b) = -\ddot{y}(t) \qquad \ldots (5)$$

where,

$$F = \mu g(1 + \theta)\,\mathrm{sgn}(\dot{Z}_b) \qquad \ldots (6)$$

$\mathrm{Sgn}(\dot{Z}_b) = \pm 1$ if $\dot{Z}_b$ is positive and negative respectively.

(c) At any instant of time during motion of the system if $|S_F| < \mu M_T g$, then the sliding of the bottom mass would stop whereas the top mass would continue to vibrate. Again the system would become single degree of freedom system and hence its motion be expressed by Eq. (1).

Parametric Study of Sliding Type Buildings

For estimating realistic forces and displacements of sliding type buildings, the response has been computed for two actually recorded severe accelerograms, viz. El Centro shock of May 18, 1940 (N-S Component) and Koyna Shock of Dec. 11, 1967 (Longitudinal component). A range of parametric values representing the physical properties of the single storey building have been used to arrive at generalised results. The results of this study are then compared with those of associated conventional non-sliding buildings subjected to same ground shaking. The various parameters of the sliding system used herein are the following:

Time period	T	=	0.04, 0.05, 0.06, 0.08, 0.10 second
Damping value	ξ	=	0.05, 0.10, 0.15
Mass ratio	θ	=	1.6, 1.8, 2.0, 3.0, 4.0, 5.0
Coefficient of friction	μ	=	0.15, 0.20, 0.25, 0.30 and 0.40

Typical results are presented in the form of acceleration response spectra in Fig. 3 in which the absolute acceleration of the top mass M_t is plotted against undamped natural period T for various parameter values by firm lines. These are termed as Frictional Acceleration Response Spectra. The acceleration response for similar conventional type single degree of freedom systems have also been plotted in the figures by dashed lines for direct comparison.

Acceleration Response

From Fig. 3 it is seen that unlike the conventional systems, 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. Only slight variation is observed for higher range of coefficients of friction. In all parameter combinations, the spectral acceleration decreases as the friction coefficient decreases. This is logical since the resistance against sliding of the system decreases as the coefficient of friction between the sliding surfaces decreases, and a build up of larger inertia force in the superstructure gets restricted. Theoretically if the coefficient of friction could be reduced to zero, no inertia forces will be transmitted to the building.

It is also seen that as the ratio of total mass to the base mass increases, the spectral acceleration decreases in all cases of parameter combinations for both the earthquakes. This means that for a given time period and damping, an increase in the top mass relative to the base mass leads to reduction in spectral acceleration.

Comparison of Response of Conventional and Sliding Systems

It is observed that the spectral acceleration of the sliding system is generally much less than that of the corresponding conventional system subjected to either of the earthquake shocks for all parameter combinations. But the sliding arrangement is seen to be of greater advantage in the case of Koyna shock where the predominant frequency and the peak ground acceleration were much higher than the El Centro shock.

TESTS ON HALF-FULL SIZE BUILDINGS

Eight specimens were tested in all, six of them being conventional that is with usual mortar joints and two with sliding joint at the plinth level. The eight specimens were tested in two sets of four each simultaneously.

Figure 4 shows the masonry dimensions, openings in walls and the position of horizontal and vertical steel bars. The mortar and reinforcement was varied in different specimens as presented in Table 1. To achieve a proper sliding joint, a reinforced concrete band was constructed at the plinth level to replace the damp-proof course as part of foundation masonry. It was finished smooth at top. Then used-up black mobil oil was painted over it and reinforced concrete band, termed as 'bond beam' for bonding the vertical steel in the superstructure masonry, was cast all round except through the door opening. The superstructure masonry was constructed on the bond beam and all vertical bars anchored to the bond beam. All specimens had reinforced concrete slab roof bonded to masonry through mortar and vertical corner steel where used was anchored into the roof slab.

Figure 5 shows the testing facility in sketch form. The shaking platform was built on an old railway wagon chasis placed on level portion of track in the middle with two striking wagons held on inclines on either side of it. The striking wagon imparted a half. sine pulse to the platform which after rolling got another half-sine pulse in the reverse direction from the other wagon. Horizontal accelerations were measured at the base and top levels of the specimens and the cracks were marked and measured. The dynamic action was increased in steps by increasing the height of fall of the loaded wagon. The strikes were applied from two opposite directions alternately.

Relative Competence of the Models to Withstand Dynamic Loads

To have a quantitative measure of the damage resisting capability of any of the model structure, two parameters are identified:

First, the 'input energy' is chosen as the parameter for defining the dynamic action on the specimens since it has the merit that the effect of all the shocks, rebounds, etc., could be taken into account by scalar addition. Second, for defining the damage level of the model, the parameter chosen is the ratio of the area of cracks in a wall to the total area of the wall in elevation. The area of a crack is equal to its length multiplied by the wall thickness. The cumulative extent of damage is plotted against cumulative input energy per unut mass for both the sets of structures in Fig. 6 from which the following significant observations are clearly made.

The extent of damage of all the models increases with the increase in input energy whether given in one big shock or a number or smaller shocks provided that each shock exceeded the damage threshold. Increase in damage of specimens 1 to 4 constituting the first set shows quite a regular trend whereas this is not so in case of the second set of test specimens 5 to 8. In the latter case, accidentally shock no. 2 became very large and under this shock, a sudden jump in the extent of damage occurred, particularly in case of specimens 5 and 6.

Considering specimens of first set it is observed in Fig. 6(a) that for a given amount of input energy, the damage is least in case of strengthened structure built in cement sand mortar, then comes the strengthened structure in mud mortar followed by the unstrengthened structure in cement mortar, whereas the unstrengthened model in mud mortar is the weakest. In the

Second set of specimens 5 to 8, Fig. 6(b) the sliding specimen 5 built in mud mortar showed less damage compared to conventional structure 6 in cement mortar and the sliding specimen 7 in cement mortar showed less damage compared to the strengthened specimen 8.

The accelerations recorded under various shocks at the base and roof of specimens 7 and 8 showed clearly that under all shocks, small as well as large, the roof acceleration of sliding specimen is much smaller than that of the conventional specimen. This observation coroborates very well with the analytical results and proves the effectiveness of the sliding arrangement in reducing the seismic response of the superstructure.

During the main shocks and rebounds it was noted that the residual displacement alternated with the alternating shocks and there was no cummlative displacement in any one direction. In an actual earthquake which in general consists of alternating pulses similar displacement behaviour is expected. Flexible connections in various service lines coming into the building will of course be required to cater for the maximum dynamic displacement.

CONCLUSIONS

The analytical and experimental studies presented above lead to the definite conclusions that the sliding arrangement provided at the plinth level with not too high a coefficient of friction will be very effective in reducing the earthquake accelerations acting on the building superstructure. Thus the new concept has the potential of not only avoiding complete collapse of masonry buildings but also minimising the cracking damage due to reduced earthquake forces and could be cheapest possible solution to the safety of masonry buildings in severe earthquake zones.

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TABLE 1 - DETAILS OF SPECIMENS (See Fig. 4)

Specimen No.	Type	Details of Mortar and Reinforcement
1	Conventional	Mud Mortar. No reinforcing
2	Conventional	Mud Mortar. Lintel band. 6mm vertical bars at corners of walls and jambs of openings set in cement.
3	Conventional	Cement Sand Mortar 1:6. No reinforcing.
4	Conventional	Cement Sand Mortar. Lintel band and vertical steel as in specimen 2.
5	Sliding	Mud Mortar. Lintel band. 6mm dia vertical bars at jambs of openings. Plinth band and Bond beam.
6.	Conventional	Cement Sand Mortar 1:6. Lintel Band. No other reinforcing.
7	Sliding	Cement Sand Mortar 1:6. Reinforcing similar to specimen 5.
8	Conventional	Cement Sand Mortar 1:6, Lintel and Plinth Bands. Vertical bars as in specimen 2.

Notes:

(i) Good burnt bricks made to half scale i.e. 112mm x 56mm x 38mm approximately used. Concrete in band and roof slab was M150 (28 day 15 cm cube strength of 150 kg/cm^2).

(ii) Lintel and Plinth bands consisted of 3 bars 6mm ∅ placed in 38mm thick concrete over all walls.

(iii) Bond beam reinforced as lintel and plinth bands.

(iv) Vertical steel at corners anchored in roof slab and foundation masonry or bond beam.

(v) Vertical steel at jambs anchored into lintel band and bond beam or plinth band.

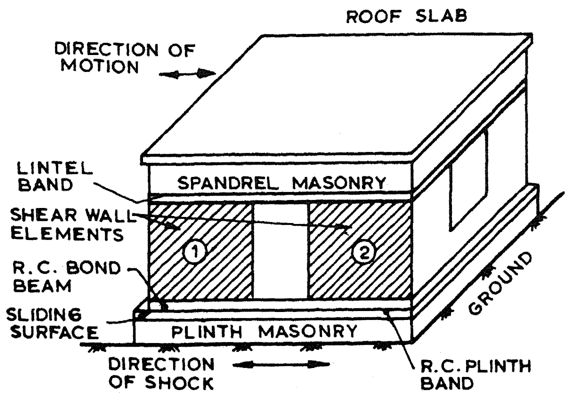
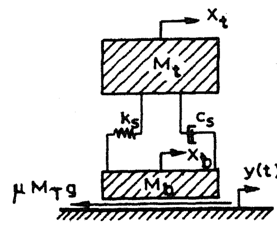


Fig. 1_ Masonry building with sliding joint

Mathematical model- Two degree of freedom system



$M_T = M_t + M_b$

$Z_b = x_b - y$ $\quad Z_t = x_t - y$

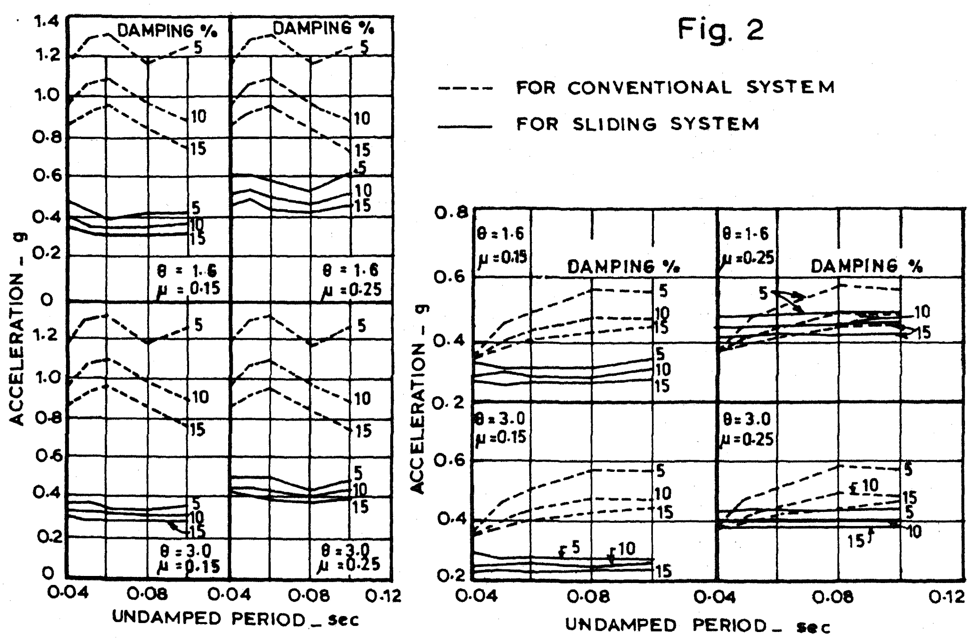
$p^2 = \sqrt{\frac{K_s}{M_t}}$ $\quad \zeta = \frac{C_s}{2pM_t}$

$\theta = M_t/M_b$

μ = COEFFICENT OF FRICTION

g = GRAVITY ACCELERATION

Fig. 2



(a) FOR KOYNA EARTHQUAKE 1967 LONGITUDINAL COMPONENT

(b) FOR EL CENTRO EARTHQUAKE 1940 N-S COMPONENT

Fig. 3 _ Frictional Acceleration response spectra (A few typical cases)

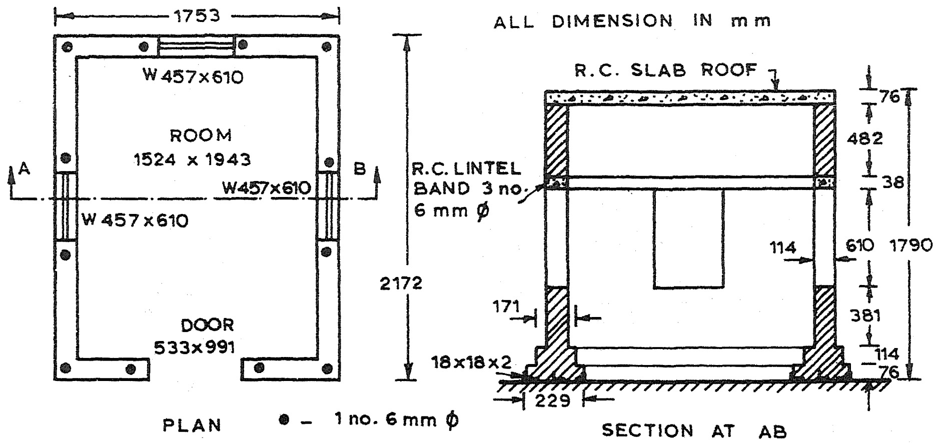


Fig. 4 _ Half size building specimens (see Table 1 also)

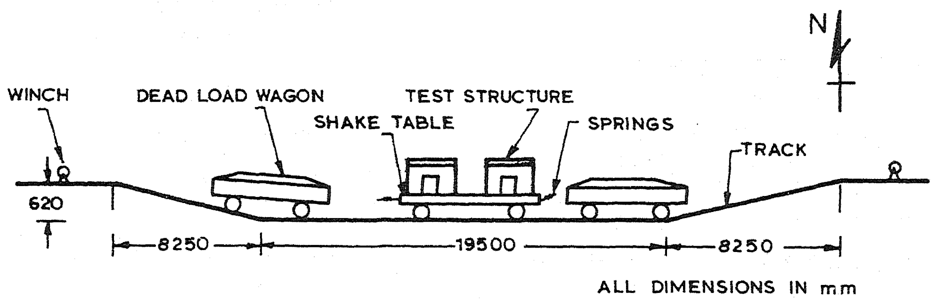


Fig. 5 _ Shake Table Arrangement

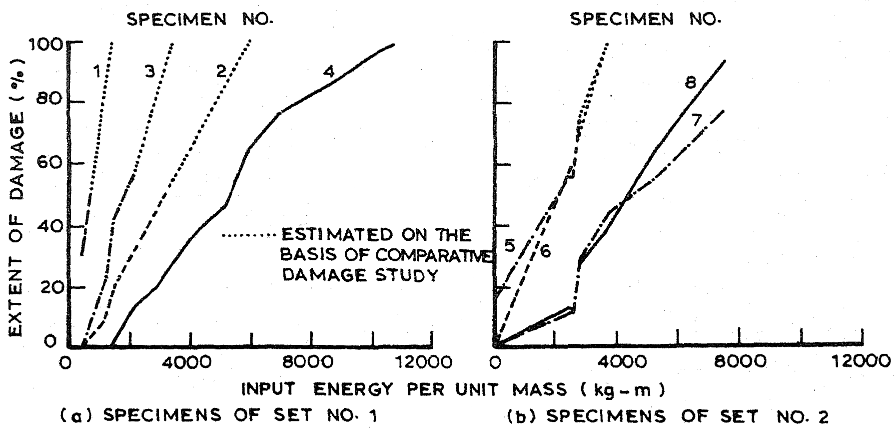


Fig. 6 _ Extent of Damage vs Input energy in half size specimens