SLIDING CONCEPT FOR MITIGATION OF EARTHQUAKE DISASTER TO MASONRY BUILDINGS

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SUMMARY

The concept of seismic isolation is applied here to masonry buildings by separating the super-structure masonry from the foundation masonry just above the finished floor level, so as to permit sliding of the super-structure during a severe earthquake in which the acceleration peaks will exceed the frictional resistance. The paper describes the details of this construction, the analytical seismic response results under the two severe earthquake accelerograms, and test results of half-size specimens under repeated severe half-sine pulses on shock type rolling-stock facility. The results show that the sliding technique is a practical and economical device to achieve non-collapse as well as less-damage masonry houses.

INTRODUCTION

It is well known that masonry buildings suffer the heaviest damage during earthquakes of moderate to severe intensities. Their vulnerability to damage results from several factors, such as, their short periods attracting large spectral accelerations, heavy weight but small tensile and shear strengths, usually poor workmanship in construction, etc. Attempt has so far been made to develop suitable strengthening methods through reinforcing steel bars placed at the critical sections (e.g. tie beam at lintel level and vertical steel at corners and junctions of walls and jambs of openings) so as to tie up all walls and piers together for ensuring integral box like action, imparting tensile strength to masonry in the tension zones and increasing the energy dissipation capability of the construction through ductile deformation of reinforcing bars(1,5,7). Shake table tests on half-size one room specimens have proved the effectiveness of such measures for achieving non-collapse construction even during severe shaking, but the walls do crack extensively before the reinforcement takes over the resisting function(9). Thus need for large scale repairs after the earthquake event is indicated even when so strengthened.

During some past severe earthquakes, viz. Dhubri 1930 and Bihar 1934 in India(6) cases were reported that small buildings which had freedom of rigid body displacement survived the earthquake while others similar but fixed at base were destroyed. A number of special seismic isolation devices have been developed in recent years by which the super-structures are connected to their foundations through flexible elements and/or energy dissipation devices are introduced (4, 8,10). The merit of the isolation system is to maintain the super-structure in the elastic hence undamaged state. This concept is quite different than the fail-safe design concept used in achieving ductility in structures for reducing the seismic response. Unfortunately, unreinforced masonry buildings of conventional construction cannot be 'economically' isolated by these techniques using isolators. A concept of

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sliding joint has been developed for such buildings to obtain the advantage of isolation(9). The main aim of the paper is to present the upto date development in this regard.

THE SLIDING JOINT CONCEPT

The sliding joint detail as conceived for masonry buildings is shown in Fig.1. The conventional construction is shown at (a) in which the plinth masonry and super-structure masonry are joined through a damp proof layer of cement-sand mortar 1:2 in 25 mm thickness or cement-sand-stone aggregate concrete 1:2:4 in 38 mm thickness. In either case water proofing compount is mixed as per specification. This layer prevents the capillary rise of moisture from the foundation masonry into the super-structure masonry. It is usual to adopt a thicker lower wall by half-brick(114 mm)than the upper wall at plinth level. This will be particularly necessary for providing a sliding joint as shown at Fig.1(b). Here the bond is broken at top of plinth masonry by plastering it smooth and laying a non-bonding membrane such as black polythelene sheet or burnt mobil oil film. On top of the membrane is cast a reinforced concrete bonding element 75 mm thick and as wide as the upper wall, reinforced longitudinally with 2 bars 12 mm dia of mildsteel or 10 mm dia of high strength deformed steel. This bonding element will be laid under all internal and external walls just above the finished floor level inside the rooms and the steel bars will have full continuity at all intersections of walls(1). Any vertical reinforcing bar in the walls,e.g., at corners and junctions of walls and at jambs of door and window openings (5) will be anchored into this bonding element. Let it be called as 'above-plinth band'. The 'band' top surface should be made rough with criss-cross marking just after laying so as to develop full bond with super-structure masonry which will be constructed on top of the 'band' by laying a mortar bed in the usual way.

Under normal conditions, the super-structure will simply rest on the non-bonding membrane and the friction will suffice to hold it in position against wind and other casual lateral forces. The 'above-plinth band'provides a laterally stiff and strong member to check against local bending of a wall in the horizontal plane and consequent vertical cracks. Also, the super-structure shall not slide under minor ground shaking so long as the peak ground acceleration ratio to gravity does not exceed the coefficient of friction. Under stronger shocks the super-structure will tend to slide and then 'above-plinth band' will tend to ensure an integral movement of the whole super-structure as one piece. The movement of the structure will follow the acceleration peaks larger than the friction coefficients and will be random in nature sometimes one way and sometimes the opposite way. So long as the superstructure continues to stay on the plinth projections, which would be about 57 mm(about a quarter brick)on either side, the super-structure wall will continue to have full bearing on the plinth. The extent of movement will depend on the earthquake intensity and the coefficient of friction and is examined later in this paper. The various pipes coming into the house from outside will have to have flexible loops as dictated by the relative displacements so as to avoid their breaking. In most developing countries, however, the most common pipe is the water supply pipe of galvanised iron and should not present too difficult a problem in this respect.

EARTHQUAKE RESPONSE ANALYSIS

Figure 2(a) shows a single storey brick building with the sliding joint at plinth level. For computing earthquake response, the building is idealised as a two degrees of freedom discrete mass model as shown in Fig.2(b). The spring action in the system is assumed to be provided by the shear walls. Internal damping is represented by a dashpot in parallel with the spring. The mass of the roof slab and of one-half the height of walls is lumped at the roof level and one-half the mass of walls is lumped at the level of 'above-plinth band'. The two-mass system is then permitted to slide at the plinth level. For analysing the system, the assumptions are made that the coefficient of friction between the sliding surfaces remains constant and that linear elastic spring stiffness is worked out by considering bending as well as shear deformation in the wall elements.

There are three stages in the motion history of the structure to be considered in deriving the equation of motions(2,9). (a) Initially, so long as the force of the moving system does not overcome the frictional resistance, mass M_b moves with the base, there is no sliding, and the system behaves as a single degree of freedom system. (b) The sliding of the bottom mass begins when the frictional resistance at plinth level is overcome by the force causing sliding. The system now acts as two degrees of freedom system. (c) At any instant of time during motion of the system if force causing sliding becomes less than the frictional force, the sliding of the bottom mass is stopped. Again the system becomes a single degree of freedom system.

For estimating realistic forces and displacements of sliding type buildings, the response has been computed for two actually recorded severe accelerograms, viz. Koyna (India) quake of Dec.11, 1967 (longi. component) and El Centro(USA) shock of May 18, 1940 (N-S component). A range of parametric values as given below presenting the physical properties of the single storey building was used to arrive at generalized results:

Time period, T	= 0.04 to 0.10 sec.
Damping value,	= 0.05, 0.10 of crital
Mass ratio, $\theta = M_t/M_b$	= 1.6, 1.8, 2.0, 3.0, 4.0, 5.0

To achieve appreciable advantage of isolation by sliding, the coefficient of friction should be considerably less than the acceleration peaks in ground motion. For Koyna earthquake (peak = 0.63g) $\mu \leq 0.4$ and for El Centro quake (peak = 0.32g), $\mu \leq 0.2$ would be reasonable. It is assumed that a coefficient of friction less than 0.15 in sliding will be difficult to obtain without using expensive materials and for a value greater than 0.40, no sliding motion may occur is most real earthquakes.

Some of the results of seismic response analysis are presented in Fig.3, in terms of the absolute acceleration of the top mass M_t related to undamped natural period T for various parameter values. These may be termed 'frictional acceleration response spectra'. The acceleration response for similar conventional fixed base single degree of freedom systems are also plotted for direct comparison. It is seen from Fig. that unlike the conventional system, the frictional spectra are generally flat and the value does 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, that is $\mu = 0.3$.

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 and a build up of larger inertia force in the super-structure gets restricted.Theoretically, if the coefficient of friction is reduced to zero, no inertia force will be transmitted to the building. Hence for given value of μ, the sliding arrangement is of greater advantage in the case of Koyna shock where the ground accelerogram peak was higher than that of El Centro.

The maximum and residual relative displacements of bottom mass are shown in Fig.4. It is seen that the residual or permanent displacement is only slightly smaller than the maximum displacement reached during earthquake motion. Therefore, the maximum dynamic displacement may be used for practical design without undue conservatism. Also the maximum value of the displacement increases as the coefficient of friction is decreased. The value of maximum displacement in the cases studied for both the earthquakes works out less than 19 mm which occurs for $\mu = 0.15$. This is considerably less than the plinth projection of 57 mm.

TESTS ON SLIDING TYPE BUILDING SPECIMENS

In order to verify the theoretical results, Qamaruddin(9) carried out preliminary tests on two small size single room specimens 914 mm x 762 mm in plan and 572 mm high (using 114x57x38 bricks) which were constructed over steel channel base so that it could either be bolted to a shake table top (fixed base condition), or unbolted and allowed free sliding movement(see Fig.5). Three coefficients of friction were used between the specimen and shake table, viz., 0.25 using graphite power, 0.34 for dry sand and 0.41 for wet sand. The shaking imparted was steady-state sinusoidal type between 8 and 26 Hz. The accelerations measured at table top and roof of specimen are shown in Table 1 along with the maximum sliding displacement. It was seen that when the base acceleration ratio to g exceeded the coefficient of friction, sliding of the specimen occurred and the roof acceleration very much decreased as compared to fixed base condition.

Table 1

Base	μ	'a' Recorded at Base g	'a' Recorded at Roof g	Ratio $\frac{a_R}{a_B}$	Slide Disp. mm
Fixed	∞	0.38	0.89	2.34	0.0
Sliding	0.25	0.32	0.20	0.63	2.0
Sliding	0.34	0.86	0.60	0.70	0.5
Sliding	0.41	0.86	0.77	0.90	0.5

For further verification as well as to study the cracking behaviour of such constructions and comparison with conventional ones, four half-size specimens of one room brick construction of sliding type have been tested on shock type railway-wagon facility along with four other similar ones but with fixity at base (3,9). The shake-platform is 7m x 6m in plan and could accommodate four specimens at one time. Thus the tests were carried out in two sets of four each. Besides the features of fixity or sliding at plinth of specimen, the mortar, wall thickness and reinforcing pattern were also varied. Figure 6 and Table 2 describe the specimens, the reinforcing used at critical sections, the acceleration at base and roof of specimens and the sliding movement in sliding-type models. The accelerations at roof and base of specimens are compared in Fig.7 and the cumulative input energy per unit mass is related to the extent of cracking damage in Fig.8(3). From the records of accelerations and cracking patterns, the following observations can be made:

a) The roof acceleration in sliding-specimens showed a more or less flat fixed value after the motion was severe enough to cause sliding whereas it continued to built-up until after sufficient cracking had occurred in fixed base specimens.

b) The extent of cracking damage was much less in sliding type than in fixed base specimens. The cracking pattern was quite different in the two types. The shear walls received severe damage in fixed base specimens but in sliding type specimens, the cross walls were more damaged. Also whereas the shear walls received mainly diagonal cracks, the cross walls got horizontal cracks.

c) The sliding movement of sliding specimens was found reversible in reversed shocks, the residual displacement being thus reduced after each complete cycle.

d) The model even in mud mortar with sliding base (Model MS) also showed extremely good resistance and it could also withstand without collapse large base shocks though with more severe damage than the specimen in cement mortar. The specimen MF did not perform so well and reached its ultimate failure at a base acceleration of only one fourth of MS.

e) The panelled wall model in mud mortar with sliding base (Model 6) also showed exceptionally good resistance against shock loads and did not collapse till the end.

CONCLUSIONS

The results of seismic response analysis of sliding type one storey masonry buildings and the experimental results of shake table tests on half-full size one storey brick specimens of conventional and sliding type, clearly demonstrated the following advantages of the sliding scheme developed herein:

a) When the acceleration peaks exceed the frictional sliding threshold, the super-structure starts sliding and the further increase of acceleration gets restricted. Thus the frictional response spectra are flat and much below the conventional response spectra. Thus good isolation effect is obtained.

b) The cracking observed in sliding specimens was much less than in conventionally strengthened specimens. A sliding specimen built even with mud mortar did not collapse.

From a practical and economic stand point also, the scheme is quite feasible requiring only usual skills of construction locally available.

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TABLE 2

TEST RESULTS OF HALF-FULL SIZE BUILDING SPECIMENS

Sl No	Description of specimen	Specimen dimension	Base Accn. at first crack	Max. Base Accn.	Corr. Roof Accn.	At max. Accn.		Obs. Sliding	
						Specimen state	U	Base Acc.	Movement
		mm	g	g	g		$(m/s)^2$	g	mm
1.	MS-LPB	l=2172	$a < 0.51$	2.15	1.22	failed	1.90	2.15	18
2.	CF-L	b=1753	$0.51 < a < 2.15$	2.15	0.75	failed	1.90	-	nil
3.	CS-LPB	h=1524	$0.51 < a < 2.15$	2.45	1.28	dmg 80%	5.18	2.45	35
4.	CF-LPVJ	t_w=144	$0.51 < a < 2.15$	2.45	1.79	dmg 80%	5.18	-	nil
5.	MF-(BWP)	l=2140	$0.35 < a < 0.62$	1.60	2.24	failed	2.63	2.63	nil
6.	MS-(BWP)	b=1840	$0.35 < a < 0.62$	6.60	1.13	dmg 95%	39.43	1.33	25
								6.60	190
7.	CS-(BWP)	h=1504	$0.62 < a < 1.32$	6.60	1.60	dmg 60%	39.43	1.33	21
								6.60	120
8.	CF-(BWP)	t_w=57	$0.62 < a < 1.32$	6.60	3.05	dmg 95%	39.43	-	nil

MS = Mud mortar, sliding type
CF = Cement mortar, fixed type
CS = Cement mortar, sliding type
MF = Mud mortar, fixed type
L = Lintel band (3 bars 6 mm ø)
P = Plinth band (3 bars 6 mm ø)
B = Bond beam just above plinth level with 3 bars 6 mm ø
V = Vertical steel at corners,(1 bar 6 mm ø)
J = Vertical steel at jambs of openings (1 bar 6 mm ø)
(BWP) = Bounded panel wall construction(as per Fig.6) using half-brick thick walls
l = Outer length, b = outer width,
h = Height above plinth, t_w = wall thickness
U = Cumulative input energy per unit mass
dmg = Damage based on total length of cracks

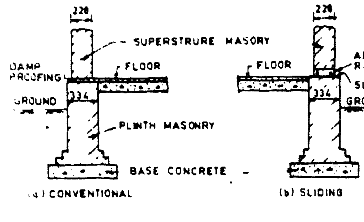

Fig 1

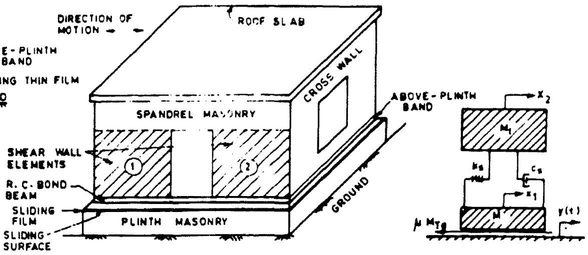

Fig. 2 _ Mathematical model for sliding system

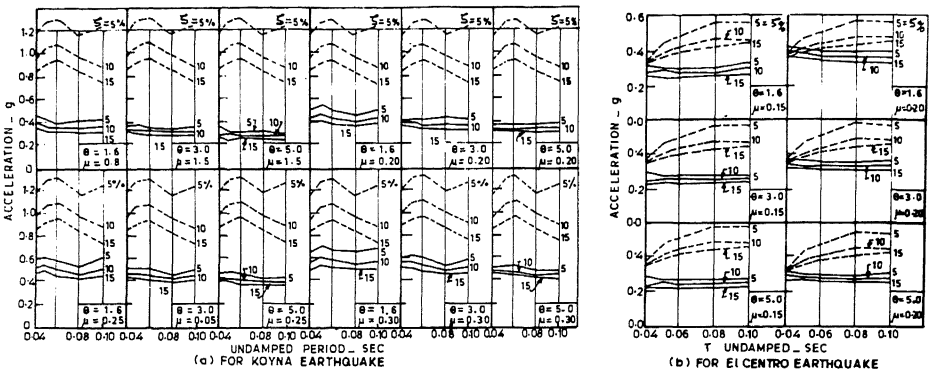

Fig. 3 _ Frictional response acceleration spectra

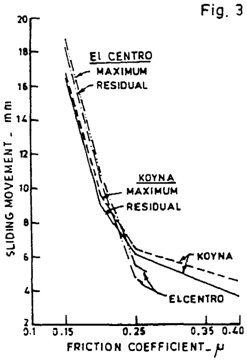

Fig. 4

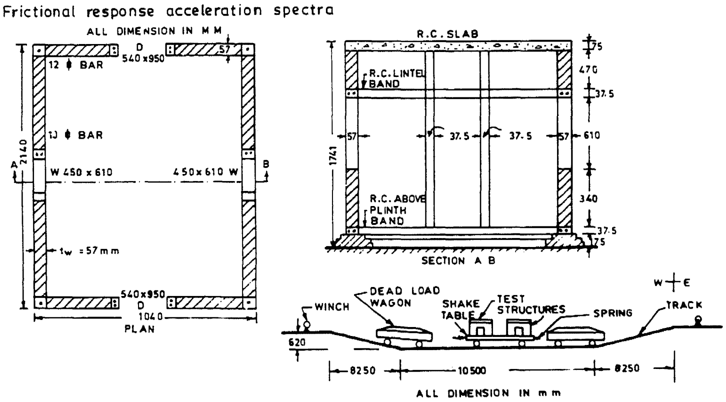

Fig. 6_ Brick specimen and rolling stock test facility

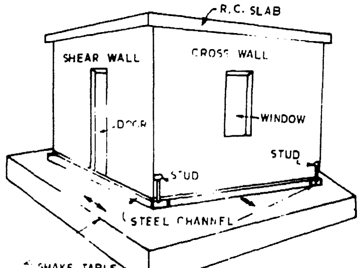

FIG 5

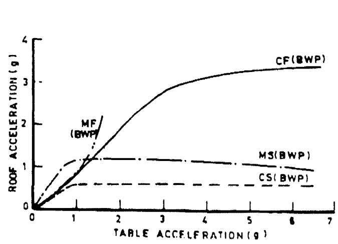

Fig. 7

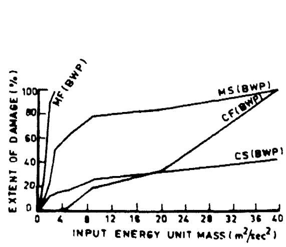

Fig. 8_ Progressive damage of wall models