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SEISMIC EVALUATION OF EARTHQUAKE RESISTANT AND RETROFITTING MEASURES OF STONE MASONRY HOUSES

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SUMMARY

Full scale models of one storeyed stone masonry houses with two different combinations of strengthening measures have been tested under progressively increasing intensity of shock on shock table facility. After the damage of models during shock table testing, these are retrofitted by existing techniques prescribed in the IS code and tested again. A three-dimensional (3D) elastic analysis of the tested structure has also been carried out by finite element method. The elastic dynamic response obtained by response spectrum method for signatures of shock and shear stresses in walls has also been computed to determine region of cracking. The codal provisions of earthquake resistance measures are effective in improving the behaviour of stone masonry models. But the cracking in the piers of walls still occur. There is a good agreement in the region of cracking in the shock table tested model with that determined by finite element analysis.

INTRODUCTION

The stone masonry buildings have proved to be the most vulnerable to earthquake forces and have suffered maximum damage. Therefore, it is necessary that realistic stone masonry houses as are being constructed in rural and hilly regions of India should be tested dynamically for evaluating various seismic strengthening measures in order to prove their effectiveness. The incorporation of retrofitting measures in existing structures is also an effective preparatory effort for reduction of damage in future earthquakes. Little research has been carried out for studying the effectiveness of retrofitting measures of existing weaker structures and damaged buildings. It is also necessary that the effectiveness of various seismic-strengthening techniques of masonry buildings may be assessed through finite element analysis because full-scale testing of realistic masonry structures is uneconomical and time consuming. The finite element analysis is considered suitable for identifying weak zones and regions of first cracking. The ultimate aim of this study is to evaluate and recommend methods for upgrad existing earthquake resistance measures.

SHOCK TABLE FACILITY AND METHOD OF EVALUATION

Shock table testing envisages testing of structure/model under impulse type of motion. The shock table facility at Department of Earthquake Engineering, Roorkee comprises of central model carrying platform on rails along with two loaded wagons on both the ends for striking and rebound (Keightley, 1977). The testing procedure consists of imparting shock of gradually increasing intensity to central wagon platform carrying the models by one of the heavily loaded end wagons. The one single shock from end wagon imparts half-sine type of pulse to central wagon. When other wagon takes the reaction, it can impart another half sine pulse from rebound. In this way one impact of end wagon can produce series of half sine pulses by reaction from end wagon. The intensity of motion of the shock table is controlled by choosing the scaled mark of the rail and spring mounted on shock table. The duration of main shock varies between 0.5 to 0.8 sec and peak acceleration of shock could be in the range of 0.45g to 2.28g. Two to three shocks are applied in each model test to bring the model to failure state. The same procedure has been employed during all the phases of testing.

The accelerations have been measured at the base of the table and roof of the model. The crack pattern is studied at each intensity of shock. Further the free vibration characteristics of original model and retrofitted model are

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also compared for studying the effectiveness of strengthening and retrofitting measures. The behaviour of each model--including the pattern of cracking, identification of weak zones, damage with progressively increasing shocks has been studied.

SCHEME OF SEISMIC EVALUATION OF STRENGTHENING AND RETROFITTING MEASURES

Two models of stone masonry houses with different strengthening measures are constructed on shock table. These models are first subjected to shock table tests and allowed to be damaged. The damaged models are then retrofitted by two different combinations of retrofitting schemes. The retrofitted models were again subjected to shock tests. The main characteristics of the models are summarized in Table 1.

Table 1: Stone Masonry Models for Evaluation of Strengthening and Retrofitting Measures

Model No.	Plan Size	Type of Roof	Strengthening Measures	Retrofitting Measures
1.	2.9mx2.6m	Gable type	R.C. Lintel band	Grouting of cracks + strengthening of wall with wire mesh+ stitching of corners walls
2.	2.9mx2.6m	Gable type	Lintel band +Corner & Jamb steel	Through stones + External bindingwith wire mesh + Ties at sill level

TYPES OF TESTS

Two types of tests have been carried out in order to obtain the necessary experimental data:

- Free vibration test for obtaining natural frequency and damping,
- Shock table test to determine the efficacy of strengthening and retrofitting scheme.

DISCUSSION OF EXPERIMENTAL RESULTS

Free Vibration Test

The free vibration tests have been conducted under three conditions (i) original model (strengthened model) under uncracked condition (ii) damaged strengthened model in cracked condition (iii) retrofitted model in uncracked condition. The ranger seismometers (SS-1) and solid state recorder have been used for recording free vibrations in the test. The results of free vibration tests have been discussed below:

Natural Frequency of Models

The natural frequency of the strengthened model in the longitudinal and transverse directions remains in the range of 2.5 to 4.5 Hz respectively. The decrease in frequency in longitudinal direction is due to decrease in stiffness because of door and window openings. The fundamental frequency of both the models has remained nearly the same, irrespective of type of seismic strengthening. After damage of models the decrease in frequency in the longitudinal direction has been observed. This is because the cracking mostly occurs in longitudinal walls due to uni-directional loading. But after retrofitting a little increase in the frequency has been noticed in comparison to the damaged model. This behaviour is obviously caused because of increase in stiffness due to addition of retrofitting elements. The natural frequency of the retrofitted models lie in between the frequencies recorded at the uncracked and cracked stages of the model. This exhibits that the retrofitting techniques are effective in restoring stiffness partially.

Damping

The damping of the strengthened model in the uncracked state lies in the range of 3% to 4%. But after damage 50% to 100% increase in damping has been observed. The cause of increase in damping is the dissipation of energy in the cracked walls. The retrofitted models have shown a decrease in damping as compared to the damaged model. This decrease may be attributed to the reduction of energy dissipation in cracks which have been well grouted during retrofitting.

Shock Table Test

The models have been tested by three successive shocks with gradually increasing intensity. The peak acceleration, duration of shock and the crack pattern of the strengthened and retrofitted models have been summarised in the Table 2.

Table 2: Response of the Strengthened and Retrofitted Models in Shock Tests

Model no.	Shock no.	Wheel position	Peak accelerations		Amplification factor	Duration of shock in seconds
			base	roof		
1.	1.	W-19	1.32g	2.00g	1.52	0.5
		rebound	0.65g	0.65g	1.00	0.6
	2.	W-20	1.60g	1.72g	1.08	0.7
		rebound	0.70g	0.90g	1.28	0.5
1R	1	W-19	1.50g	2.30g	1.54	0.6
		rebound	0.60g	0.80g	1.33	0.4
	2	W-20	1.71g	0.90g	0.52	0.5
		rebound	1.30g	0.90g	0.68	-
	3	W-21	2.25g	-	-	0.2
2	1	W-19	1.38g	2.30g	1.66	0.7
		rebound	0.80g	1.00g	1.25	0.5
	2	W-20	1.76g	1.10g	0.63	0.8
		rebound	1.17g	0.90g	0.76	0.7
	3	W-21	2.28g	1.50g	0.66	0.4
		rebound	1.06g	0.94g	0.88	0.3
2R	1	W-19	1.40g	0.94g	0.67	0.5
	2	W-20	1.90g	-		0.5
	3	W-21	2.45g	-		0.2

Table 2 shows that acceleration pattern along the height of the model, prior to cracking, has been represented by an inverted triangular shape but the pattern became irregular with progressively increase in the extent of deformations. After cracking the acceleration at roof level generally becomes small as compared to base. This indicates that the damaged lower part of the model functioned as a kind of base isolator which prevented propagation of energy into upper part. It can also be seen that with the increased damage to the models the amplification factor (ratio between the maximum acceleration at the top of the model to the maximum acceleration of the shock table) has decreased.

Table 2 also exhibits that the acceleration at roof level in model 1R under first shock has been amplified upto 80% but in model 2R it has been de-amplified upto 60% because grouting in model 1R has improved the integral behaviour of the model. A decrease in acceleration at the roof during successive tests has been noticed in comparison to base acceleration. This phenomenon is due to cracked regions of the wall as a result of the previous test. The cracked lower portion of the wall acts as isolation element, that causes reduced acceleration at the roof level. In model 2R the scheme used in retrofitting has proved to be effective for integrating the structure and reducing further cracking. But it may not be helpful in regaining the inner strength of the model. Therefore, the grout injection is necessary before undertaking a retrofitting scheme for a damaged model.

Damage Index

The comparison of seismic resistance of strengthened model with retrofitted model have been done on the basis of damage index as defined by Tomazevic et al (1992). The damage index is calibrated in four parts : (i) First damage to the walls; 0.25 (ii) shear cracks in the walls; 0.50 (iii) severe cracks, crushing of corners, falling of parts of the walls; 0.75 (iv) Collapse; 1.00. The damage index of the original model and the retrofitted model has been shown in Table 3.

Table 3: Damage Index of Models under Different Shocks

No. of Model	Shock No.	PBA	Damage Index (Id)				Total Damage	ER measures
			NSW	SSW	ECW	WCW		
Model 1	1	1.32g	0.25	0.25	0.25	0.00	0.25	Cement mortar + lintel band
(strengthened)	2	1.60g	0.25	0.50	0.50	0.25	0.50	
Model 1R	1	1.50g	0.00	0.00	0.00	0.00	0.00	Grouting + wire mesh + stitching at corners
(retrofitted)	2	1.71g	0.25	0.25	0.00	0.00	0.25	
	3	2.25g	1.00	0.50	1.00	0.50	0.75	
Model 2	1	1.38g	0.00	0.00	0.00	0.00	0.00	Cement mortar + lintel band + corner & jamb steel
(strengthened)	2	1.76g	0.25	0.50	0.25	0.25	0.25	
	3	2.28g	0.75	0.75	0.50	0.25	0.50	
Model 2R	1	1.40g	0.00	0.00	0.00	0.00	0.00	Bond stones + external binding + ties at sill level
(retrofitted)	2	1.90g	0.25	0.25	0.25	0.25	0.25	
	3	2.45g	0.75	0.50	0.50	0.25	0.50	

The damage index has been calculated for each wall of the model as well as for the total damage of the model. Table 3 shows that the damage in model 2 is 25% less in comparison to model 1 at the same excitation. In model 1R there has been no damage under first shock in either of the walls while in the original model it has been recorded as 0.25. As a result of further shock W-20, the damage has been reduced up to 50% as compared to the original model. This suggests that the retrofitting scheme of model 1R not only helps to regain the strength but also increases the strength and stiffness of the model. In successive shocks the original model was unable to bear further shock but in the retrofitted model a greater intensity of shock (W-21) has been applied and around 75% damage has been observed. The retrofitting scheme employed for model 2R has been effective in regaining the original strength of the structure. The extent of damage of the model 2R is nearly similar as compared to the original model

Failure Mechanism of Models

The model building considered herein may be treated as typical box system in which the walls (with door and window openings) parallel to excitation force are called inplane walls or shear walls, whereas, the walls (gable walls) perpendicular to them are termed as out-of-plane walls or cross walls. During testing inplane walls induce bending moment which is maximum at its lateral ends producing vertical cracks. The diagonal cracks in the walls in shear failure are due to the principal tensile stresses which develop in the walls because of vertical and lateral loads. Due to the door and window openings the effective width of the shear wall is decreased resulting in smaller shear strength. The shear becomes critical in the wall and failure occurs in the form of diagonal cracks.

Behaviour of Strengthened Models

In model 1, the damage to the shear wall is limited upto the lintel level, the damage has also occurred at the corners. The portion of the load bearing longitudinal walls below sill level have failed in shear. The failure state of model 1 is partial collapse of longitudinal walls below lintel level and damage at corners (Figure 1). In model 2 cracking has occurred between lintel and sill level (Figure 2).

Figure 1: Crack Pattern in North Shear Wall of Model 1 under shock W-20, PBA= 1.60g

Figure 2: Crack Pattern in North Shear Wall of Model 2 under shock W-21, PBA= 2.28g

Model tests of stone masonry structures indicate that the damage started from the corners of model and the corners of door and window openings. Hence, it may be suggested that strengthening of corners will not only improve the lateral resistance capacity significantly but will also improve energy dissipation without much strengthening of wall piers. Thus, the failure pattern of models in random rubble masonry shows that, with the increasing earthquake resistance measures the extent of damage has been greatly minimized under same intensity of shock.

Behaviour of Retrofitted Models

The retrofitted model 1R has shown marked improvement in performance with a total absence of cracks in retrofitted portion and in other region of wall only minor cracks have occurred. On the other hand, stitching of walls has proved to be extremely useful since the stitched portion has not at all damaged further under during higher intensity shocks (Figure 3).

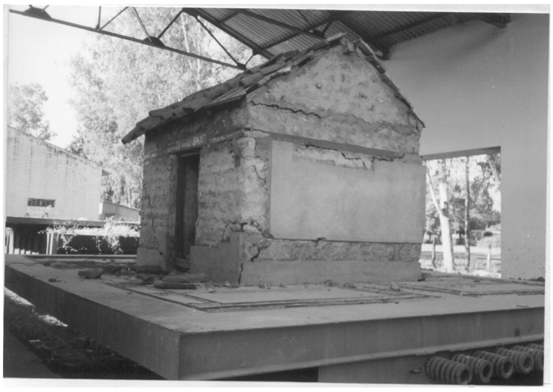


Figure 3: Crack Pattern in South Shear Wall of Model 1R under shock W-20, PBA= 1.60g

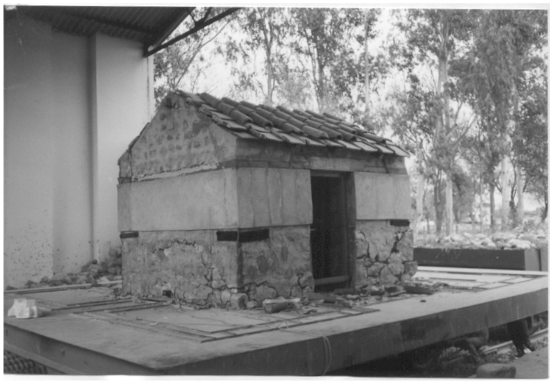


Figure 4: Crack Pattern in South Shear Wall of Model 2R under shock W-21, PBA= 2.28g

In retrofitted model 2R damage has been caused to the structure below sill level. The most interesting feature is that the cracks in the retrofitted regions of the model are almost absent. Cracks have developed at other places in all the four walls. The major cracks in the southward shear wall have occurred at the bottom of the right pier which are almost diagonal. The eastward and westward cross walls have undergone minor damages since only one crack has developed below the sill level (Figure 4).

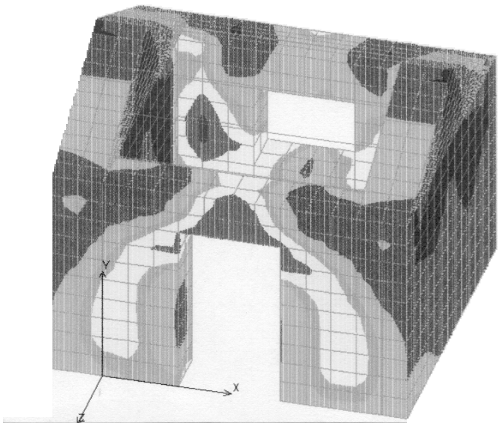
RESPONSE SPECTRUM ANALYSIS OF MODEL SUBJECTED TO SHOCK TABLE MOTIONS

A three-dimensional elastic analysis of tested structure has been carried out by finite element method with the following assumptions (i) The masonry is considered as homogeneous and isotropic (ii) The behaviour of masonry is assumed to be linear elastic upto stress level in the uncracked state (iii) The model is considered fixed at the base. This approach is used to determine weak zone and region of cracking in the structure. The weak regions imply that zone where there is a concentration of maximum shear stresses. The region where shear stress exceed the permissible value designates the region of first cracking. The COSMOS/M programme has been used with 8 noded brick element for walls and roof has been modeled with 3D truss element. A total number of 2277 nodes and 1403 elements have been used in the model of the tested structure. The strengthening measures have been simulated by increase of Young's modulus (E) in the region of lintel band.

The elastic dynamic response has been obtained by response spectrum method for signatures of shock obtained from experiment. The acceleration response spectra has been computed for each motion recorded at the base of the model for 5% damping. The resulting responses are obtained by combining modal responses of ten modes of vibration using square root of the sum of squares (SRSS).

The finite element model has been used to determine the magnitude and directions of shear stresses in walls for shock table motion of W-19 and W-20. The earthquake stress were combined with stresses due to gravity loads. The analysis has been carried out for unstrengthened as well as strengthened models. The maximum shear stresses (τxy) and average shear stresses in the longitudinal shear wall in unstrengthened model under shock W-20 are 0.17 MPa and 0.14 MPa respectively. In strengthened model the maximum shear stress and average shear stress in walls are 1.0 MPa and 0.1 MPa respectively. Although maximum shear stresses in strengthened model is larger as compared to unstrengthened model but the average shear stresses in the pier of walls are reduced.

The maximum shear stress contours for the northward and southward shear walls have been obtained from response spectrum analysis. It can be observed that in unstrengthened model the maximum shear stress are concentrated in the regions above lintel level, mid portion in piers of shear wall and around the opening (Figure 5). In strengthened model the maximum shear stresses are concentrated at the lintel level (Figure 6). This shows that cracking will occur in these regions. There is a good agreement in the region of cracking in the shock table tested model with the results of FEM analysis.



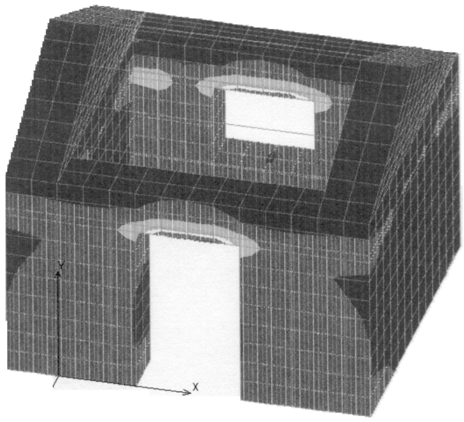


Figure 5: Shear Stress Coutour of Unstrengthened Model under Shock W-20

Figure 6: Shear Stress Coutour of strengthened Model under Shock W-20

CONCLUSIONS

On the basis of experimental testing and analytical studies for seismic behaviour of masonry buildings, several conclusions have been drawn. Some of the important conclusions are highlighted:

1. The codal provision of earthquake resistance measures such as lintel band, roof band, corner and jamb reinforcement are effective in improving the behaviour of stone masonry models: the cracks are reduced, corner separation does not occur, roof does not collapse. But the cracking in the piers of walls, below the lintel band, still occur.

2. On the basis of shock table test the following recommendations are made to reduce the damage in stone masonry model (i) an integrated roof system with shear connection with walls (ii) band at sill and lintel level (iii) extra strengthening at corners in the form of vertical bar and dowels (iv) strengthening around the door and window openings.

3. The 3-D elastic finite element analysis of masonry model can predict dynamic behaviour and the region of cracking in the structure. It accounts for both inplane and out-of-plane action in the walls and can capture the key features of the response and provide a fairly close prediction of the distribution and severity of damage.

4. The injection of cementitous grout on localized damaged areas can restore the original strength and stiffness. The introduction of external horizontal tie bar to a wall can increase the strength and ductility of the model. Moreover, welded wire mesh in damaged region not only increases the lateral resistance of the wall but also prevents shear and flexure failures of the models.

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