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Effectiveness of wooden bond beams in dry stone masonry houses

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Stone masonry houses are the most common type of construction in the Alpine Himalayan Belt across Pakistan, India and Nepal. However, the seismic resistance of these houses is highly questionable if constructed without any form of lateral support. In this paper, the effectiveness of wooden bond beams as a retrofit solution has been examined. Dry stone masonry houses have been modeled by finite element method considering stones as linear solid and interfaces as joint elements. The joints are allowed to open and slide satisfying the Mohr-Coulomb criteria. To calibrate the values used in the numerical modeling an experiment using a small scale wall made of wooden blocks was shaken in small custom made table. The corresponding parameters which showed good agreement with experimental results were taken as inputs for the non linear dynamic analyses of various model houses. The results showed that wooden bond beams can be an effective technique for upgrading low strength masonry houses in low seismicity regions

Key Words: *Masonry, bond beam, joint element, dynamic analysis*

1. Introduction

Stone has long been used as a building material for the construction of houses due to its availability and durability in the Alpine Himalalyan Belt (Pakistan, India, Nepal etc.) Large settlements of stone masonry buildings constructed with lime or earth mortar and even without mortar can be found in the area. Stone walls are built by stacking stones over stones normally in two leaves. Vertical joints are avoided as far as possible by placing various sized stones alternately. Corner stones are chisel dressed and mid span stones are hammer dressed. In cases where long stones are not available to break the vertical joints, wooden pieces are used. Roofing material may vary depending on location and can be corrugated galvanized iron sheet, slate or thick rammed earth laid over wooden joists and battons. One type of the dry stone masonry constructions using wood as bond beam called Hatil[1)] in Turkey and Bhatar[2)] in Pakistan (Fig.1) consists of single storey random rubble masonry with horizontal wood beams at specific intervals. Similar dry stone masonry houses fairly chisel dressed but without bond beams, are found in western Nepal also. Houses are generally one to two storeys and may have multiple rooms added at different stages of their history. Storey height is typically 2m and houses have small doors and windows and can therefore be dark inside even in day time. People use firewood to cook their meals, and stone built houses are highly fire resistant and durable against environmental degradation. Locally trained masons can build these houses easily and materials are locally available making these types of construction economical. If built correctly, they perform well under vertical loads. However, due to distinct directional properties of stone with its irregular shapes, it is difficult to make stone walls strong against lateral loads. Lateral strength depends upon friction between the stones and a very low cohesive strength of mortar if it is used. Thus, these houses have sustained heavy damages in historic earthquakes and have been the primary cause of fatalities. Usually, dry walls consist of two leaves of stones with total width of approximately 45 cm. The leaves have very weak bonding and interconnectivity and can become unstable even in minor earthquakes. One possible methods of upgrading the wall could be to encourage the use of wooden elements as used Hatil[1)] in Turkey and Bhatar[2)] in Pakistan. Spence and Coburn[1)] found wooden elements to be effective in mitigating against failure in their experimental investigations. Cao and Watanabe[3)] analyzed brick masonry buildings by finite element methods considering bricks as solid elements and mortars as viscoelastic joint elements providing opening and sliding phenomenon. The buildings were analyzed before and after retrofitting by timber frame and they were found to be an effective way of preventing failure.

Wood bond beams have been found to be effective in resisting earthquake induced forces, hence these are known as seismic bands. However, the extents to which they contribute to preventing failures have yet to be investigated. There have only been a few studies testing with small dry walls examining these constructions[4,5)]. Here, an attempt has been made to evaluate the performances of these houses in earthquakes. Two typical types of houses (one and two rooms) with and without applied wooden bond beams are modeled considering stones as elastic elements and interfaces between stones as inelastic joint elements. Subsequently, three dimensional dynamic analyses were carried out by applying various seismic accelerations obtained from past earthquakes and the results are discussed in the concluding part of this paper.

Fig.1 Contemporary Bhatar construction, Tarand-NWFP-Pakistan[2)]

2. Numerical Modeling

Even small masonry houses are made of thousands of pieces of stones and have joints at least three times that number. These joints are weak and are found to deform first under any kind of loading and govern the overall behaviour and failure mechanism of these structures. A large numbers of factors such as interior voids, irregular shaped units, varying properties of stone to stone, quality of workmanship contribute to making the behaviour of these walls very complex. Computational approaches to investigate the strength of masonry have been conducted by various methods, ranging from simplified methods to highly sophisticated method which uses interface or joint element to define the possible failure[3,6-13)]. Analyses using distinct element method (DEM) which is also considered as a simplified micro method, model the wall as an assembly of small blocks and interfaces. The contact forces and displacements at the interfaces of stressed assembly of blocks are found through evaluation of equations obtained from Newton's law of motion which defines the movement of the blocks. There have been two recent pieces of work using DEM in examining the behavior of buildings: Papantonopoulos et al.[6)] investigated the efficiency of DEM to predict earthquake response of classical monuments by comparing the numerical results with experimental data. Alexandris et al.[7)] investigated collapse mechanisms of non-engineered stone masonry houses subjected to severe earthquake excitations using distinct element method. Two and three dimensional analyses of two types of buildings were studied. They concluded that two dimensional analyses were unable to simulate realistic responses. The out of plane failure of the long wall was found to be the dominant mode of the failure mechanism in stone masonry.

On the other hand, various investigators have utilized finite element method (FEM) which uses elastic and inelastic interfaces between units called discontinuities as having properties of sliding and separation[8-13)]. Zienkiewicz et. al.[8)] proposed a joint element for the laminar nature of a material which is confined by a narrow zone such as an old fault surface or joint in rocks. An isoparametric joint element in two and three dimension was introduced[9)] to represent the interface between shell and solid elements. The stiffness matrix of the joint element was formulated considering interface as separate isoparametric element with zero thickness. The non linear behaviours of joints were characterized by slip and separation taking place at the joint plane. Separation of joints was considered when it became tensile. Recently, this concept has also been used in modeling brick masonry[10)-12)] to simulate time dependent sliding and separation along mortar joints. Three dimensional finite element models were formulated by considering the relative displacements between the top and the bottom of base elements and the constitutive relationship, based on material properties containing shear and normal stiffness which can be found from stress displacement curves of the mortar. A brick masonry wall was analyzed in static and dynamic loadings and was found to be capable of predicting appropriate responses[12)].

There have also been experimental investigations done in dry joint cut sawn stone masonry walls subjected to in plane and combined loadings[4)] and similar small walls have been investigated analytically in monotonic and reversed cyclic loadings[5)] considering multi-surface interface model[13)]. From the literature review, we found that most researchers have followed the idea of modeling the units as solid elements and interfaces as zero thickness joint elements. Thus, the same approach considering stones as solid elements and their interfaces as joint elements has been employed here.

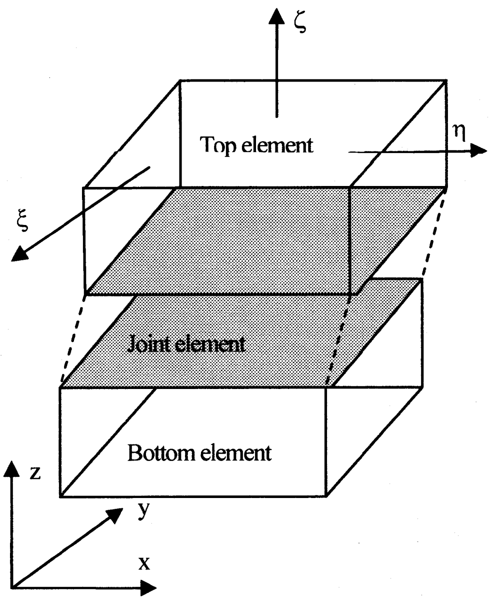


Fig. 2 Formulation of solid and joint elements

Generally, stone walls consist of a large numbers of irregular size stones, and modeling each individual stone and their interfaces in their as-built condition would be impossible. Thus a simplified numerical model has been developed making an equivalent group of eight node elastic solid elements for stones and eight node joint elements for their interfaces as shown in Fig. 2. In the Fig., x, y and z are global axes and ξ, η and ζ are local axes. The ultimate objective of this dynamic analysis is to solve the widely known equation of motion:

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=-[M]\{\ddot{u}_g\} \tag{1}$$

where, [M], [C], [K], are mass, damping and stiffness matrices, $\{\ddot{u}\}$, $\{\dot{u}\}$ and $\{u\}$,are acceleration, velocity and displacement responses respectively and $\{\ddot{u}_g\}$ is input ground acceleration.

The stiffness matrix for the system is obtained by assembling individual solid and joint element matrices. The formulation of the stiffness matrix for solid elements is referenced in Chandrapatla and Belgundu[14]. The displacement of joint elements depends on the relative movements of the top and bottom solid elements (Fig. 2), and the corresponding stiffness matrix for zero thickness joint elements can be formulated[8)-10)] as shown in equation 2.

$$[K]_j^e=\left[\int_{-1}^{1}\int_{-1}^{1}[N]^T[k][N]\det[J]d\xi d\eta\right] \tag{2}$$

$$[k]=\begin{bmatrix} k_{sx} & 0 & 0 \\ 0 & k_{sy} & 0 \\ 0 & 0 & k_n \end{bmatrix} \tag{3}$$

where, k_{sx}, k_{sy} and k_{sn} are components (shear stiffness along x direction, shear stiffness along y direction and normal stiffness) of material property matrix [k] of joint, [N] and [J] are shape function, and Jacobian matrices, and ξ and η are local coordinates.

Normal and shear stiffness are calculated by regarding the wall as a series of two vertical springs, one representing the stone unit and the other representing the joint which leads to the following formula[4].

$$k_n=\frac{1}{h\left(\dfrac{1}{E_{wall}}-\dfrac{1}{E_{unit}}\right)} \tag{4}$$

$$k_s=\frac{k_n}{2(1+\nu)} \tag{5}$$

where k_n is normal stiffness of joint, k_s is shear stiffness of joint, h is height of unit (average height of stone unit), E_{wall} is Young's modulus of elasticity of wall, E_{unit} is Young's modulus of elasticity of unit and is taken equal to 15,500N/mm^2, and υ is Poisson's ratio (assumed equal to 0.2).

The modulus of elasticity is dependent on many factors such as type of stone, workmanship, void inside the wall etc. A wide range of values have been proposed in literature[15] varying from 200-1000 N/mm^2. In situ tests were carried out in Faial Island, Azores[16], and the modulus of elasticity of random rubble stone masonry wall was found to be 200N/mm^2. This value corresponds to 1.3% of the modulus of elasticity of stone and has been used in this study.

In order to get the damping matrix (equation 6), the mass and stiffness proportional to Rayleigh damping has been used.

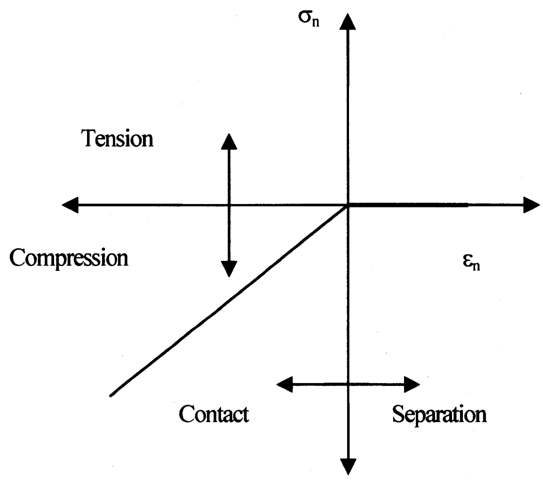
$$[C]=\alpha[M]+\beta[K] \tag{6}$$

where, α and β are coefficients selected to control the damping ratios of the lowest and highest modes expected to contribute significantly to the response.

Unfortunately, there is a sever lack of data available on damping parameters in linear solid mechanics problems, and even less information is available on damping in non linear dynamic analysis. Tzamtzis and Asteris[12] did dynamic analysis of brick masonry wall by using quite high damping coefficients and found that the numerical simulation was matching with experimental results. At multiple modes of vibrations, damping ratios change with natural frequencies because of different mass participation factors at different modes[11]. For the problem under consideration the coefficients α and β have been taken as 0.0555 and 0.0105 respectively so as to maintain initial value of damping [11),16)] 6% and maximum value 10% considering dry masonry constructions are highly deformable.

3. Constitutive Relationship

The joint is characterized as fully elastic, perfectly plastic and incapable of taking any tensile forces. The idealized constitutive relationship shown in Fig. 3 has been used to denote the sliding and opening of joint. Separation occurs when the normal strain is greater than zero and since the joint cannot take any tensile stress and both act in the normal direction, the shear stiffness has also been set to zero. Contact occurs when normal strain is less than zero, and normal forces are assumed to be restored corresponding to the normal stiffness of the joint. Sliding occurs when the shear at joints exceeds the value given by the Mohr-Coulomb yield criterion (equation 7).



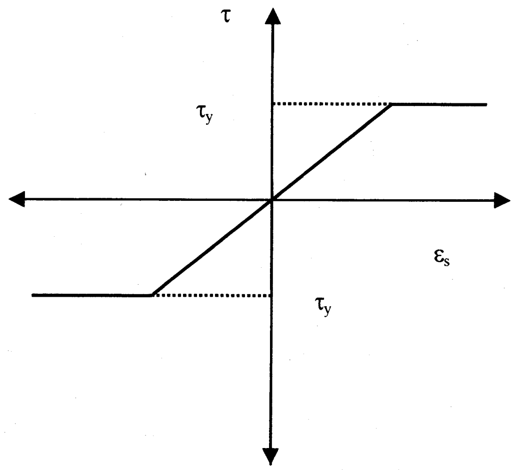


Fig. 3 Constitutive relationships for joints in normal (top) and shear (bottom)

$$\tau_y = c + \sigma_n \tan\phi \tag{7}$$

where, τ_y is yield shear stress, c is cohesion (equal to zero being mortar less joint), σ_n is normal stress and $\tan\phi$ is coefficient of friction.

4. Calibration of parameters

Stiffness parameters represent the strength of the joint. In the case of brick masonry, it is calculated from the relationship between wall thicknesses, mortar thickness and the modulus of elasticity of bricks[10]. In the case of dry stone masonry there is no material between the two stones, therefore the stiffness of joint can be zero to infinity depending upon the way of thinking. In order to investigate this, a small experiment was done with wooden blocks.

Wooden blocks were cut into pieces and a dry wall (0.40mx0.08mx0.26m) was constructed as shown in Fig. 4. The wall was shook manually on a small table using a handle. The acceleration at the base was measured by means of an acceleration sensor and the final displacement was measured. The unit weight of wood was 4.47 KN/m^3, the modulus of elasticity [18] was taken as 8100000 KN/m^2 and Poisson's ratio as 0.3. The stiffness (k_n=2883430 KN/m^3, k_s=1201430 KN/m^3) were calculated using equations 5 and 6 assuming the modulus of wall was 1.3% of the unit. The dry wall structure is a discontinuous, highly deformable system and is similar to dry stone masonry houses, therefore the same Rayleigh damping coefficients (α=0.0555, β=0.0105) were assumed. The coefficient of friction was simply measured by putting a block over flat base and raising the base gradually. When the block started to move angle was noted and average of tangent of values was found 0.3. Our ultimate goal is to analyze the LSM house, thus parameters have been taken focusing appropriateness for stone masonry houses. Purpose of calibration analysis is to see the suitability of parameters itself. Finally, a finite element model of the wall considering the wooden blocks as solid and interfaces as joint elements was made.

Using the above mentioned parameters and an acceleration time history (Fig. 5) obtained from a previous shaking test, a dynamic analysis was carried out. The displacements obtained from the experiment (Fig. 6) and numerical simulations (Fig. 7) were plotted. A comparison of displacements measured along height of the model wall has been plotted as shown in Fig. 8. Due to the inconsistent friction between elements and possibly human error, this is considered inevitable. However, the average displacements obtained in the experiment of the two edges are in good agreement with the numerical result. The numerical simulation gives 3.4 cm displacement at the top and the average (left edge and right edge) displacement of the experimental wall was also 3.5 cm.

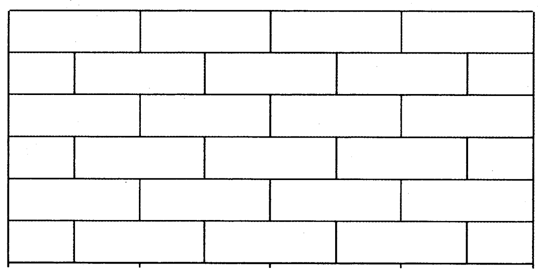


Fig. 4 Wood block wall (0.40mx0.08mx0.26m)

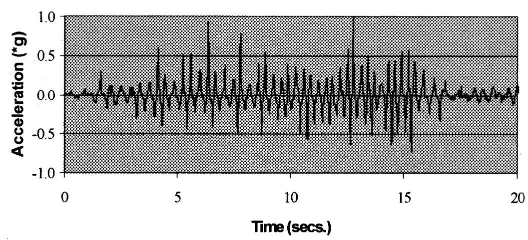


Fig. 5 Measured acceleration at the base of wall

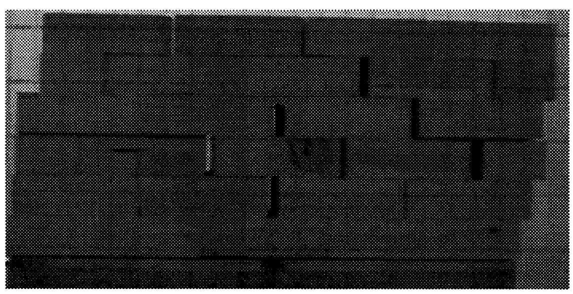


Fig. 6 deformed wood wall after experiment

In discontinuous systems, behaviour of individual element affects the overall response. However, measured residual deformation at the end of the test is consistent with numerical result though positions of individual elements are not consistent. That is because of different frictional values among the elements. But in numerical analysis, same friction coefficient was employed for all elements. Though movement of individual elements and energy dissipation at each time step have effects in overall response, it is not practical for big models which may have thousands of elements like stone masonry house which are dealing here. So, the deformations of individual elements have not been measured at each time step.

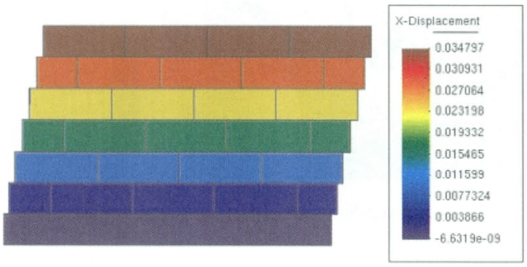


Fig. 7 Simulated displacement of wooden wall

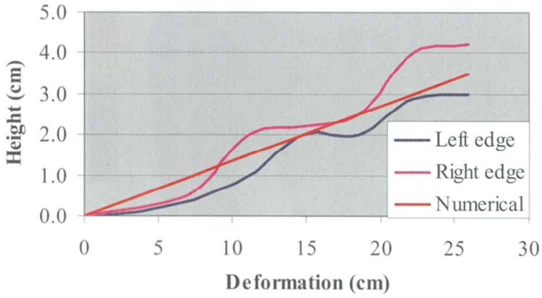


Fig. 8 Comparison of displacements

5. Analysis of masonry houses

Two typical types of single storey houses, one with a single room with an internal size 4.05mx3.15m and the other with two rooms of equal internal sizes of 3.15mx3.60m and both with wall thickness 0.45m were modeled. The roof load depends on what type of roofing is used. Approximate calculations suggest a roof load equal to 1.5KN/m^2 which represents a thin slate roof, would be a good average considering roofs could also be made of thicker slate, corrugated galvanized iron (C.G.I) sheets or thick rammed earth. Both houses were analyzed twice with and without applying wood bond beams and subjected to different ground motions. At first, static analyses were run considering self weight and roof loads. The stresses and strains obtained from static analyses were used as initial values for dynamic analyses. In dynamic analyses, roof loads were converted into masses by dividing acceleration due to gravity and allocated as lumped masses at the top nodes of walls. Material properties for wooden bond beams and coefficient of friction were taken same as used in calibration analysis. Peak accelerations of the three components of the time histories for the Kobe earthquake of 1995 and the 1940 El Centro earthquake are given in Table 1.

The aim of this analysis is to see whether these houses can sustain large deformations. Therefore, solid elements have been assumed linear and the focus is in the non linear deformation at joints. The properties of the solid and joint elements are shown in Table 2.

Table 1, Peak accelerations

Earthquake	Kobe, 1995			El Centro, 1940		
Peak acceleration (m/sec^2)	X	Y	Z	X	Y	Z
	8.06	5.87	3.36	2.63	2.62	2.63

Table 2, Various models

SN	Description	Elements
1	One room dry stone house Element size 0.45mx0.45mx0.30m	Stone:328 Joint:586 Total:914
2	One room dry stone house with wooden bond beams Stone element :0.45mx0.45mx0.26m Wood element :0.45mx0.45mx0.08m	Stone:328 Wood:170 Joint:766 Total:1264
3	Two room dry stone house Element size:0.45mx0.45mx0.30m	Stone:524 Joint:951 Total:1475
4	Two room dry stone house with wooden bond beams Stone element: 0.45mx0.45mx0.26m Wood element :0.45mx0.45mx0.08m	Stone:524 Wood:272 Joint:1238 Total:2034

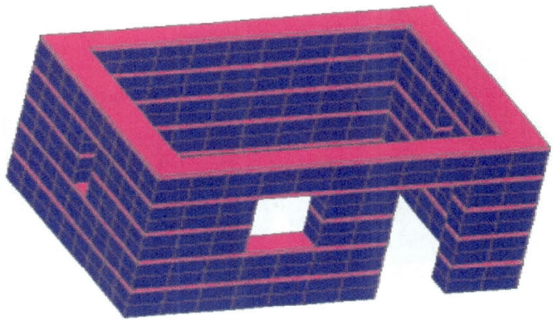


Fig. 9 Single room house with wood bond beam

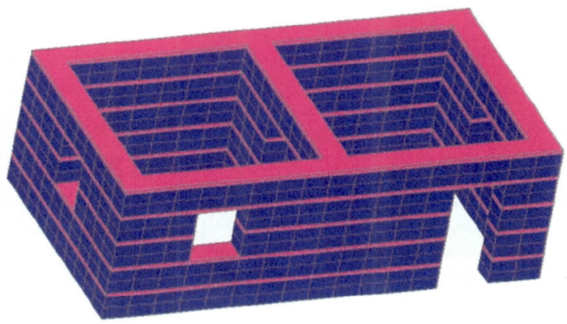


Fig. 10 Two room house with wood bond beam

Four different models with varying sizes of elements, numbers of rooms and material used were prepared:

1. Model 1 is single storey, one room house with a lintel beam over the opening. The lintel beam is assumed to have the same properties of as the stone.
2. Model 2 is similar to model 1 but has horizontal wood bond beams at 0.52m interval.
3. Model 3 is single storey, two room house with lintel beam above opening.

4. Model 4 is the same as model 3 but has wood bond beams added at 0.52 m intervals.

The details of the numbers of elements, sizes etc. are shown in Table 2. The model houses are shown in Fig. 9 and 10 in which the pink colored continuous elements represent the wood bond beams and the blue elements are stone elements and vertical and horizontal lines are joints.

6. Discussion

The four models for the two houses described above were analyzed using various ground motions and their deformations and stresses are plotted in Figs. 11-22. Initially, model 2 was analyzed using the Kobe earthquake time history. The acceleration was too high for the masonry building and the house produced large deformations (Figs. 11 and 12) within 8 secs. The reason why displacements were much larger along the x direction as compared to the y direction was due to the large accelerations in that direction (Table 1).

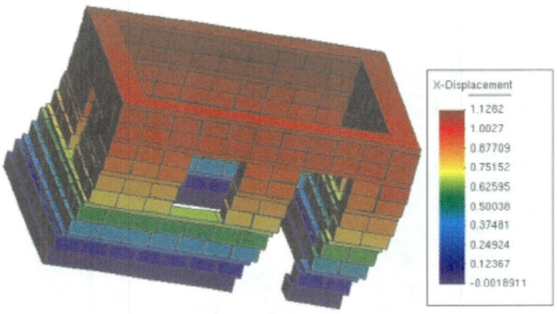


Fig. 11 Deformations of model 2 house in Kobe earthquake

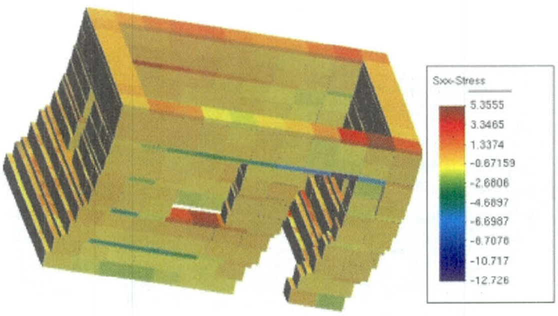


Fig. 12, Stresses in model 2 house in Kobe earthquake

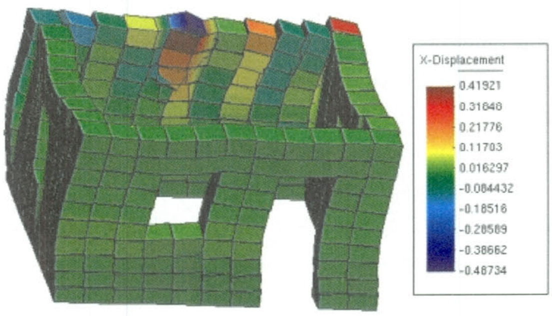


Fig. 13, Deformations in model 1 in El Centro earthquake

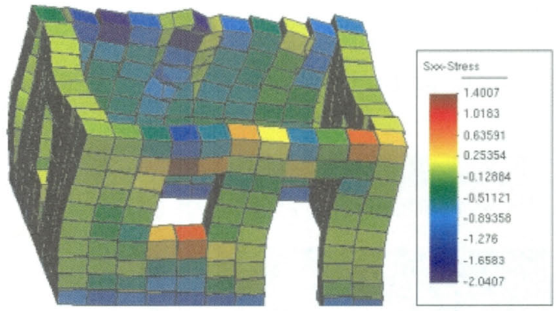


Fig. 14 Stresses in model 1 El Centro earthquake

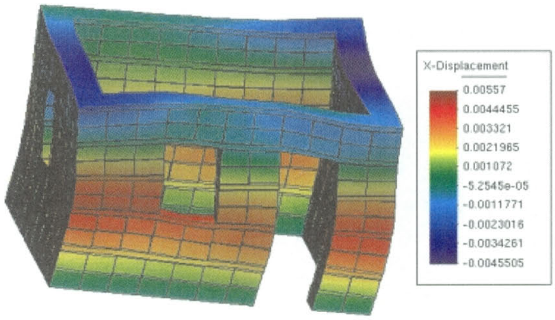


Fig. 15 Deformations in model 2 in El Centro earthquake

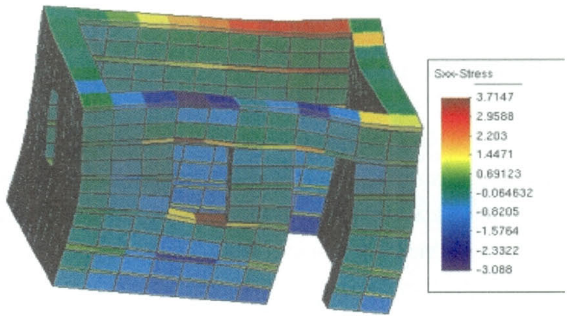


Fig. 16 Stresses in model 2 in El Centro earthquake

In all of the analyses, when deformation exceeded 30 cm, the program automatically stopped due to the large deformations. The limit 30cm is arbitrarily assumed value. It can be less or more. Even if it is not assigned the program runs to final step. It can simulate beyond this limit also, however, as the displacement increase the nonlinear iterations also increase and it directly elongates the computation time. The stone masonry houses are very weak and cracks are formed and become unserviceable even in few centimeters residual deformations. Average length of random rubble stone used in masonry house is less than 20cm. If residual deformation exceeds 30cm, most of the stones are dislocated from its original position and the house is no more usable. Our aim is not to look whole collapse process. If we are looking for effectiveness of wood bond beam, the deformation of such houses under seismic loadings should be less than the few centimeters.

The stresses in all Figs. are in ton/m^2 and displacements are in meters. Model 1 does not have any seismic band and is weaker than model 2, thus it will be meaningless to analyze using the input motions of the Kobe earthquake. Therefore, model 1 was analyzed in El Centro earthquake. The deformations and stresses are shown in Figs. 13-14. It sustained large deformations quickly. In Fig. 13, we can see deformations were higher in the back wall than in the front. This is because rigid lintel beams were placed over the openings, which did not deform and strengthened the walls and eventually, the house failed due to the separation of the weaker wall. In model 2, which was analyzed with the El Centro ground motion, the structure was found to perform well as shown in Figs. 15-16. In order to test the performance limit, model 2 was analyzed again with an amplification of 2 of the El Centro ground motion; this deformed with large displacements (Figs. 17-18). Subsequently, the response of the two room house as represented by model 3 was analyzed subjected to the El Centro earthquake ground motion.

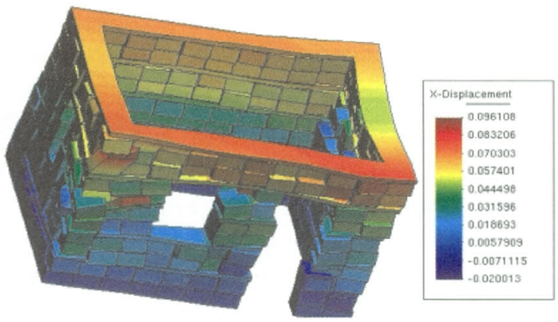


Fig. 17, Deformations in model 2 in 200% El Centro earthquake

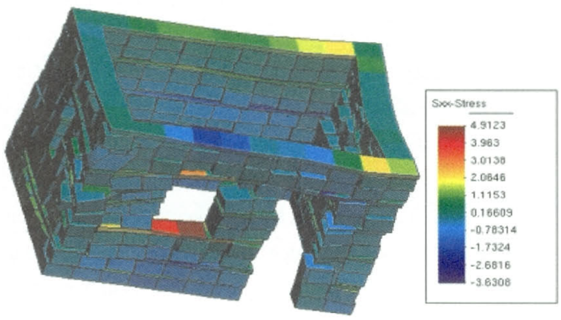


Fig. 18 Stresses in model 2 in 200% El Centro earthquake

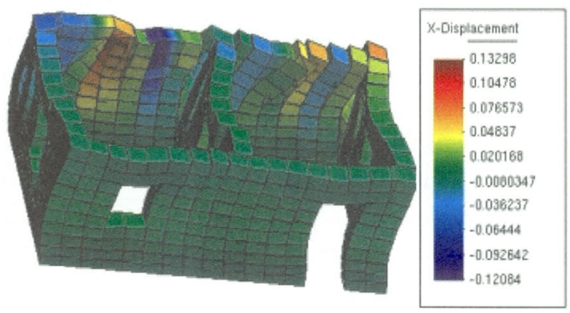


Fig. 19 Deformations in model 3 in El Centro earthquake

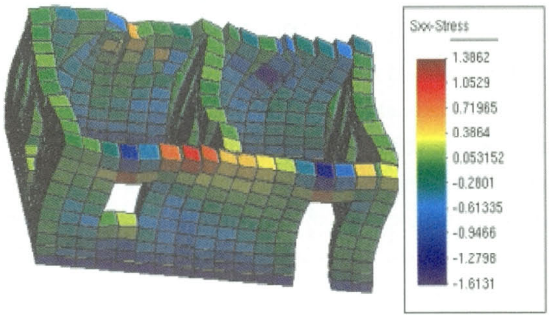


Fig. 20 Stresses in model 3 in El Centro earthquake

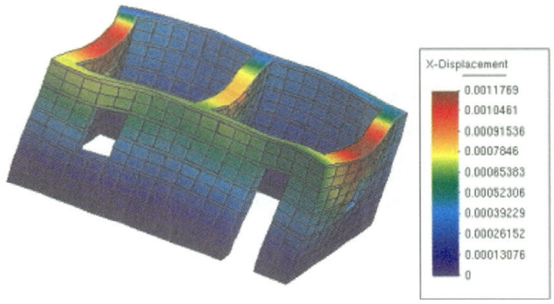


Fig. 21, Deformations in model 4 in El Centro earthquake

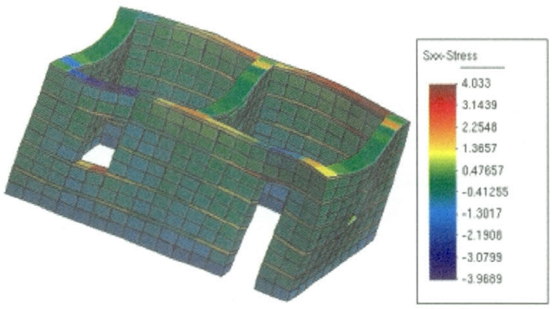


Fig. 22, Stresses in model 4 in El Centro earthquake

To examine the response of a two-room house, model 3 was analyzed using the El Centro earthquake motion. Like in model 1, model 3 also sustained large deformations and failed due to separation of the back wall (Figs. 19-20). Model 4 building was analyzed using the same input motion and was found to perform well (Figs. 21-22) like model 2. In conclusion, both model 2 and model 4 houses which had been strengthened by wood bond beam deformed less than 6mm and showed good performance under earthquake loading.

Stone blocks considered in the numerical analysis consist of equivalent block of many different sized stones. If the stone sizes vary each other, moduli of elasticity of walls also vary. It directly affects the value of stiffness constants. As irregular sized stones increase, wall becomes weaker and spring constants are less than that of regular sized stone wall. However, blocks should be as small as the average size of stone in the wall. In these simulations, the wall was divided making the length and breadth of the element equal to the width of the wall and the height equal to nearly half of the length. Width of wall has been taken as the reference for size of elements. In order to see size effect, the elements were further subdivided and analyzed. The differences of deformations are negligible but computation time increased by far. However, even a small house consists of thousand of units; and each unit will possess different properties and shapes and therefore show different behaviour. In this regard this model may be still too generic. If the elements are again further divided into small elements, computation time would be too long and is therefore governed by the level of accuracy required. Thus, size of elements considering the width of wall can give reasonable response.

We analyzed four models from two typical types of houses under various ground motions. If a stronger earthquake such as the Kobe earthquake is expected, wooden bond beam alone would not be able to resist the collapse of these houses. Under slightly lower acceleration levels, such as when the houses experienced an acceleration level twice that of the El Centro event, the houses still sustained very large deformations and failed. In the El Centro earthquake, the peak acceleration was about 0.31g and testing under this input ground motion, the houses without wooden bond beams still failed, but houses constructed with the additional wooden beam did not.

The main possible failure mechanism for stone masonry houses are skin splitting, vertical cracking at corners, separation of wall, wedge shape failure and diagonal cracking. If we looked at the figures we can find most of them in this study also. In Figs. 11-12, houses failed in shear and swept to collapse. Kobe 1995 earthquake's acceleration is so high that LSM house can not resist. In usual practice, lintel beams (Figs. 13-14, 19-20) are often provided over the openings even in unreinforced LSM houses. Lintel beam is not provided at back side wall and it is the weakest one. Thus corner cracking starts at the junction of two walls leading to large deformation. This may be reasonable as more than forty percent houses had damaged in El Centro earthquake. In Figs. 17-18, two times amplified El Centro 1940 was given, corner and diagonal cracks have formed near openings leading to collapse, which is quite natural since area around openings are the weakest zones. Both one and two roomed wooden beam reinforced houses (Figs. 15-16, 21-22) performed well under El Centro 1940 earthquake. The key point here is that the coefficient of friction was taken as 0.3 and the peak acceleration was 0.31g. Theoretically, sliding should not occur until 0.3g therefore the question arises as to why model 1 failed since it had equal friction. One possible explanation is that the house in model 1 experienced tension first, which the caused separation and there was nothing between the stones to control the tension leading ultimately to failure, even though it still had spare shear capacity. The wood bond beams modeled here assume rigid connections at their edges, where elements are linear and have negligible deformation which in turn confines the wall and the stone elements. Forces at the joints develop where there are relative displacements. Wood beams break vertical joints, hold the corners effectively and join the two wythes which are responsible for diagonal cracks, separation of wall and forming vertical cracks at corners and splitting of wall into two folds respectively. A single wood beam connects many stone elements and is therefore responsible for controlling the deformation of many joints.

Displacements generated by models 2 and 4 were low. There are 3 possible explanations for this:

1. Firstly, there are no precise and predetermined values of permissible displacements or drift for these types of houses to define a credible failure mechanism. Very small tensile forces could lead to collapse of these structures because of bulging, which is the prominent failure mode of double-leaved stone masonry walls. There is no bonding between the elements and although the presence of a wood beam could reduce the deformation substantially, the rest of the wall not attached to the wood beam could still fail due to tensile forces at the joints.
2. Secondly, this may be due to the method of modeling. In a real wall, there are many stone blocks between the two beams but in this model there are only two blocks along vertical direction. Therefore, each block would be in contact with the wooden beam, either at the top or the underside. The faces of the joint in contact with wood would deform less. Thus, the model may underestimate deformation as only one joint is free to move in the numerical analysis which is not true in the case of a real wall. If the houses are modeled with thousands of elements and joints the computation time is very long.
3. Lastly, in this analysis solid elements have been assumed linear and only large deformations at interfaces have been examined. Non linear deformation in solid elements can be significant in small deformations when the wall is confined by the wood bond beam.

To the authors' best knowledge; this was the first attempt at analyzing this type of dry stone masonry house in detail. Using similar method, few studies[3), 10)] have been done in brick masonry. They are bonded by cement sand mortar and dry stone masonry houses are quite different from brick masonry. This method has been implemented in Pakistan[2)] as disaster mitigation measure, but nobody has investigated dry stone houses analytically, numerically and experimentally. Does it resist any earthquakes? How much peak acceleration can be resisted when wood bond beams are used? These questions have been addressed here, following the numerical methods that can be found in open literatures. However, the problem which we dealt is totally new and investigation presented here is novel.

7. Conclusion

The behaviors of two types of dry stone masonry houses under various ground motions were investigated through detailed dynamic analyses. From these analyses, it was clear that dry stone masonry houses strengthened by applying wood bond beams would not be able to resist strong earthquakes such as Kobe 1995. However, it can be an effective technique in confining the walls under smaller ground motions similar to the El Centro earthquake of 1940. The small deformation obtained from the analyses shows that sufficient cracks could develop and make the houses inhabitable after earthquakes; however, the wood bond beams would prevent complete collapses of the dry stone masonry houses which would ensure life safety in low intensity earthquakes Thus, this could be an appropriate upgrading technique in low seismicity zones along the Alpine Himalayan Belt where these houses are common and frequent but low acceleration earthquakes are expected. Since wood can be locally available, it is the only most economical solution for upgrading these kinds of vernacular houses.

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