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Effect of containment reinforcement on the seismic response of box type laterite masonry structures – an analytical evaluation

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Abstract. Laterite blocks are used for construction of masonry walls since ages in the South-western coastal areas of India. The south-west coastal areas of India lie in zone III of seismic zonation map of Indian code IS 1893-2002. In spite of the fact that laterite is the most favored masonry material in these regions of India, the structural performance of laterite masonry has not been systematically investigated. Again there are no previous studies addressing, in detail, the seismic performance of laterite masonry buildings. Now that these areas are becoming more and more important from point of view of trade and commerce, there is a need for a detailed research on the seismic response of laterite masonry structures located in these areas. The present paper reports the results of such a study of the seismic response of box-type laterite masonry structures. Time history analysis of these structures under El-Centro acceleration has been performed using commercial finite element software ANSYS. Effect of 'containment reinforcement' on the seismic response of box type laterite masonry structures has been evaluated.

Keywords: laterite masonry; box-type structure; finite element analysis; natural frequencies; seismic response; time-history analysis

1. Introduction

The term "laterite" was first proposed by Buchanan in 1807 to describe the reddish ferruginous, vesicular, unstratified and porous material with yellow ochres occurring extensively in Malabar, India. In India, laterites occur in the states of Kerala, Karnataka, Goa, Maharashtra, Tamil Nadu, Andhra Pradesh, Orissa, Bihar, Assam, and Meghalaya. The word "laterite" is derived from the Latin word "later" meaning brick (Gidigasu 1974). Laterites are extensively used as building blocks in buildings, especially in south-western coastal areas of India (IS 3620-1979). In spite of a widespread use of laterites in buildings, no systematic research study has been undertaken on this extensively used material - on its engineering properties, particularly on the strength and durability aspects (Kasturba 2005). Again large regional variations have hindered an in-depth study to characterize laterite as a masonry material (Sutapa Das 2008).

According to IS 1893-2002, south west coastal areas of India come under Seismic zone III. In this region, laterite is the most popular material for the construction of both load-bearing and

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partition walls in single and two storeyed structures. It is also used for the construction of in-fill walls in framed structures. Hence an understanding of the seismic response of laterite masonry structures is required.

A wall subjected to lateral out-of-plane forces behaves as a plate, bending in two directions. The bending in the vertical direction (normal-to-bed-joints) causes horizontal cracks, while the bending in the horizontal direction (parallel-to-bed-joints) causes vertical cracks. Horizontal RCC bands are provided to prevent the growth of vertical and diagonal cracks in masonry elements apart from acting as a beam at the openings. The growth of horizontal cracks can be prevented by the provision of vertical reinforcement along the height of the wall. Structurally it is more efficient if the vertical reinforcement is provided at the surface of the walls where flexural strains are higher (Raghunath 2003). Embedding such vertical reinforcement bars at the edges of the wall piers and anchoring them both in the foundation at the bottom and in the roof-band at the top, delay the X-cracking and enhances the capability of wall piers to resist horizontal earthquake forces. In this study, effect of vertical 'containment reinforcement' provided on both the faces, on all the walls at a regular spacing, on the seismic response of single storeyed box-type laterite masonry structures has been investigated in detail.

2. Literature review

Karantoni and Fardis (1992) have studied the behaviour of and damage inflicted on stone-masonry buildings during the Kalamata earthquake, Greece, of 1986. Based on finite element analysis results, out-of-plane bending and the transfer of out-of-plane lateral loads to the transverse walls have been reported as the cause of most of the damage. They have concluded that finite element analysis can be used to improve understanding of the seismic behaviour of masonry buildings and to develop and assess techniques of seismic strengthening.

Bruneau (1994) has made a number of observations on the seismic performance of unreinforced masonry buildings (URM). He has discussed different types of failures such as: lack of anchorage between floor and walls, anchor failure when joists are anchored to walls, in-plane failure, out-of-plane failure, and failure under combined in-plane and out-of-plane bending effects. It has been emphasized that URM buildings are most vulnerable to flexural out-of-plane failure. Bruneau has also pointed out that when masonry constructions of poor quality often show total failure, monumental/institutional masonry buildings of high quality often perform quite well.

Andreaus (1996) investigated the failure criteria of masonry panels under in-plane loading, which has been attributed to three simple modes: slipping of mortar joints, cracking of clay bricks and splitting of mortar joint, and middle plane spalling. A specific example is worked out to show as to how to apply the criteria to predict the failure load and failure mode of a particular masonry panel. The discussion is confined to the in-plane behaviour of solid-brick, single-wythe masonry and does not consider the effects of out-of-plane bending produced by eccentrically applied loads or lateral instability.

Raghunath *et al.* (2000) have carried out studies on the ductility of brick masonry walls with containment reinforcement. Containment reinforcement consists of thin ductile wires provided on both faces of masonry wall, held together with the help of lateral ties provided through the bed joints. Characterization of static and dynamic behavior of unreinforced masonry and masonry provided with containment reinforcement is done. Initially unreinforced masonry walls were tested to obtain their strength and elastic properties. Later, brick masonry units provided with containment reinforcement, were tested to obtain moment-curvature relationships. Containment reinforcement has not only increased ductility but has also resulted in increasing the ultimate moment capacity.

Thakkar and Agarwal (2000) have conducted a seismic evaluation of earthquake resistant and retrofitting measures of stone masonry houses. It has been reported that model tests of stone masonry structures indicated that the damage started from the corners of model and the corners of door and window openings. Hence, it was 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. On the basis of shock table tests the following recommendations were made - an integrated roof system with shear connection with walls, bands at sill and lintel level, extra strengthening at corners in the form of vertical bars and dowels, and additional strengthening around the door and window openings.

Tavarez (2001) has discussed three techniques to model steel reinforcement in finite element models for reinforced concrete - the discrete model, the embedded model and the smeared model. In the discrete model, the reinforcement is modeled with bar or beam elements that are connected to concrete mesh nodes. Therefore, the concrete and the reinforcement mesh share the same nodes and concrete occupies the same regions occupied by the reinforcement. A drawback to this model is that the concrete mesh is restricted by the location of the reinforcement and the volume of the steel reinforcement is not deducted from the concrete volume. The embedded model overcomes the concrete mesh restriction because the stiffness of the reinforcing steel is evaluated separately from that of the concrete elements. The model is built in a way that keeps reinforcing steel displacements compatible with the surrounding concrete elements. However, the additional nodes required for the reinforcement increase the number of degrees of freedom, and hence the run time. In the smeared model, the reinforcement is assumed to be uniformly distributed over the concrete elements. The properties of the material model in the element are constructed from individual properties of concrete and reinforcement using composite theory. This approach is used for large models where the reinforcement details are not essential to capture the overall response of the structure.

Jagadish *et al*. (2003) have studied the behaviour of brick masonry structures during the Bhuj earthquake of January 2001. They have reported that (i) higher bond strength improves the earthquake resistance of masonry (ii) use of lintel band, seems to introduce a rigid box-like behaviour in the upper portion of the buildings while the portions below the lintel bands cracked badly suggesting the need for more horizontal bands at different levels (iii) provision of corner reinforcement in corners and junctions, as suggested by IS 4326-1993, has to be properly bonded with the surrounding masonry possibly with dowels or keys to prevent separation. According to them the horizontal bands might not be adequate in strengthening against out-of-plane flexure, especially for flexural cracks that run horizontally. In order to prevent out-of-plane flexural failure and to improve the ductility of masonry walls, reinforcement in the form of 'containment reinforcement' has been recommended. Raghunath (2003) has reported dynamic analysis of brick masonry structures with such containment reinforcement.

Bakhteri *et al*. (2004) numerically verified the results of experimental investigations on the effect of mortar joint thickness on compressive strength characteristics of axially loaded brick-mortar prisms. Micro-modeling with two different material assumptions has been attempted. In one, both phases of the materials are replaced with an equivalent homogeneous material with derived elastic properties and the other treats the masonry as a composite material consisting of the brick and the mortar. Composite material model gave more accurate prediction of the stress distribution in the prisms and hence this model was more appropriate than the homogeneous material model. Large discrepancies between experimental and FEM results has been reported in FEM model with mechanical properties taken from the experimental study, confirming that the properties of mortar inside the joints are different from the properties of mortar cubes.

Saikia *et al*. (2006) have studied the effect of provision of RC bands on the dynamic behaviour of masonry buildings. Stress analysis of typical masonry buildings, with and without RC bands, under lateral static and dynamic loads have been discussed. The natural frequencies and mode shapes of buildings with and without bands have been presented. They have concluded that although reinforced concrete bands enhance the structural integrity by contributing to the connectivity of walls, they are not adequate in preventing out-of-plane collapse of some segments of wall. Need to develop simple methods of providing vertical reinforcement in these regions has been stressed.

Gumaste *et al*. (2007) studied the properties of brick masonry using table-moulded and wire-cut bricks of India with various types of mortars. The strength and elastic modulus of brick masonry under compression were evaluated for stiff-brick/soft-mortar and soft–brick/stiff-mortar combinations. Both prisms and wallettes were studied. In western countries, brick masonry generally consists of bricks which are stronger and stiffer compared to the mortar used. On the contrary, bricks of India show relatively lower strengths and elastic moduli. The state of stress developed in brick and mortar components of masonry depends on their relative elastic properties. When bricks are relatively softer than mortar, if the brick-mortar interface bond remains intact until the failure of masonry, the brick will be under triaxial compression and mortar will be under uniaxial compression and bilateral tension. In such a scenario, the failure of masonry is initiated by the tensile splitting of the mortar in the joint. The mortar failure will then extend to the brick causing masonry failure.

Vyas and Reddy (2010) have developed a three dimensional non-linear finite element model based on micro-modeling approach to predict masonry prism compressive strength and crack pattern of solid block masonry. The FE model uses multi-linear stress-strain relationships to model the non-linear behaviour of solid masonry unit and the mortar. Masonry prism compressive strengths predicted by the proposed finite element model are about 19% less than the experimental values.

Sahin A. (2010) has developed a simple assistant program named ANSeismic, for implementing earthquake analyses of structures with ANSYS and SAP2000, finite element codes. ANSeismic is free software program and can be used as a tool in time history analysis of structures by researchers. Structural system is constructed in ANSYS by using GUI or APDL. The seismic records are then loaded from PEER Strong Motion Database and earthquake analysis files are produced in ANSYS or SAP2000. The structural models constructed in ANSYS may be analyzed by just loading the analysis file developed with ANSeismic. SAP2000 time history source data file may also be produced with ANSeismic.

3. Natural frequencies of box type laterite masonry structures

The forces attracted by a structure during an earthquake are dynamic in nature and are functions of ground motion and the dynamic properties of the structure itself. The response of the building, within its elastic limit, is mainly dependent on both the frequencies of the ground motion and the natural frequencies and mode shapes of the building. It is hence essential to obtain the natural frequencies and mode shapes of buildings in order to understand their response to earthquake induced ground motions (Raghunath 2003). In south-west coastal areas of India, laterite box type structures are constructed either with light roofing components such as tiled roof or with a rigid RCC slab for the roof. Light roofing components generally rest loosely on the walls. Such buildings with light roofs and low rigidity can almost be idealized as a building without a roof (Raghunath 2003). Herein single storeyed box type laterite masonry structures, both without and with roof were analyzed for their free vibration response characteristics.

Box type laterite masonry structures were modeled in ANSYS (Version 10). Typical plan dimensions of 6m x 3m x 3m (L x B x H) have been considered for the buildings analyzed here (Raghunath 2003). Provisions of openings in the buildings have been considered in accordance with IS 4326:1993 guidelines with one door opening (of size 1.0m x 2.1m) and one window opening (of size 1.0m x 0.9m) on one longer wall and two window openings on the other longer wall. Each of the short walls is assumed to be provided with one central window opening. All the walls of the building were assumed to be fixed at their bases, all along their lengths. The top nodes were free in structures without roof, and merged to roof nodes, in structures with roof. Vertical 'containment reinforcement' made of 12mm diameter steel bars was provided on the surface of the walls on both the faces at a spacing of 1m. Laterite masonry structures were meshed using eight noded three-dimensional solid elements (SOLID45) of dimensions 0.2 m x 0.2 m x 0.23 m, each node having three translational degrees of freedom. The vertical reinforcements were modeled using truss elements (LINK8) of length 0.2 m. LINK8 element of the ANSYS library is a uniaxial tension-compression element, also with three translational degrees of freedom at each node. The bars of steel reinforcement were modeled using the discrete model (which is generally used to model steel reinforcement in finite element models for reinforced concrete), as given by Tavarez (2001).

While masonry is quite often modeled with orthotropic material properties, in the present study, it was assumed to be isotropic and values of the modulus of elasticity of laterite masonry obtained from experimental results were made use of. Table 1 gives the properties of masonry, RCC and reinforcement bars used in the finite element analysis. Details of mechanical properties of laterite and laterite masonry have been presented in Sujatha *et al*. (2011).

Eight types of single storeyed box type laterite masonry structures of same geometry as given in Table 2, of same geometry of walls were selected for the analyses. Configurations of the eight types of structures analyzed are shown in Figs. 1 and 2.

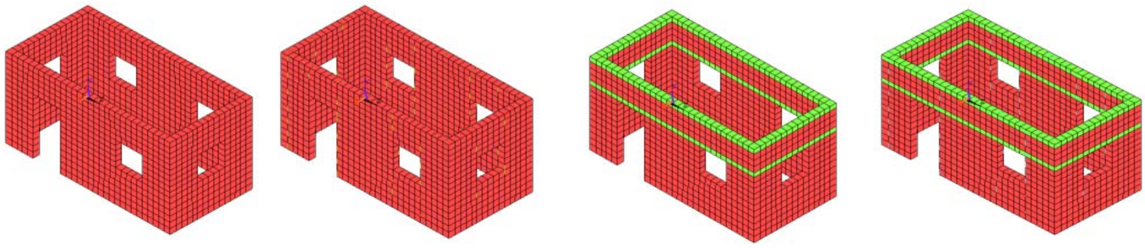
Initially free-vibration analyses of the eight types of single storeyed box type laterite masonry structures listed above were conducted to determine the natural frequencies and the corresponding mode shapes. The first ten frequencies obtained thereof for all the different structures analyzed are presented in Table 3 and Table 4.

Table 1 Material properties used in the FE analysis

Property	Laterite masonry in 1:6 cement mortar	Reinforced concrete	Vertical reinforcement
Modulus of elasticity (GPa)	1.2	25	200
Poisson's ratio (assumed)	0.15	0.15	0.3
Mass density (kg/m^3)	2500	2500	7850

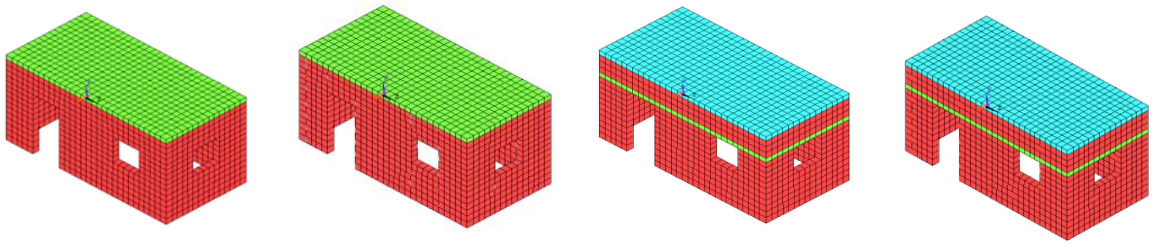
Table 2 Types of single storeyed box type laterite masonry structures analyzed

No.	Type	Description of the structure
1	A	Unreinforced laterite masonry structure without roof
2	AV	Laterite masonry structure without roof and with containment reinforcement
3	ALR	Laterite masonry structure without roof, with lintel band and roof band
4	ALRV	Laterite masonry structure without roof, with lintel band, roof band and containment reinforcement
5	B	Unreinforced laterite masonry structure with roof
6	BV	Laterite masonry structure with roof and containment reinforcement
7	BL	Laterite masonry structure with roof and lintel band
8	BLV	Laterite masonry structure with roof, lintel band and containment reinforcement



(a) Type A (b) Type AV (c) Type ALR (d) Type ALRV

Fig. 1 Configurations of box-type laterite masonry structures without roof

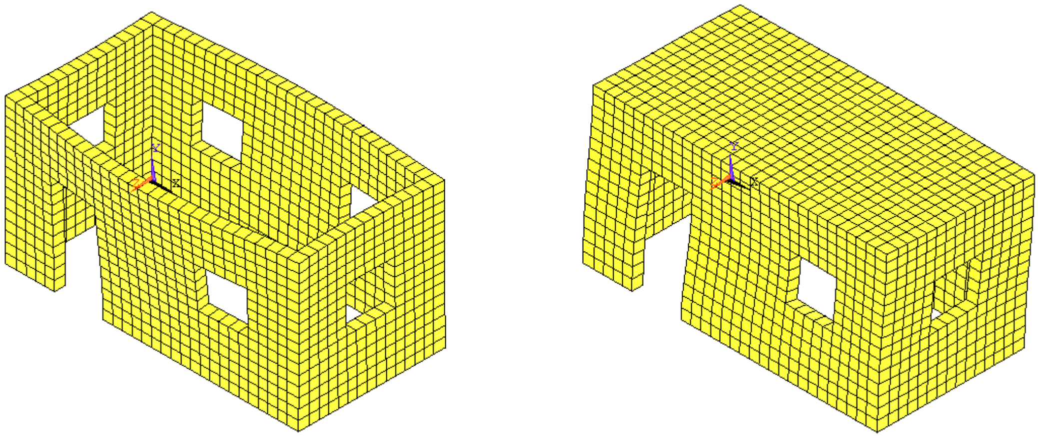


(a) Type B (b) Type BV (c) Type BL (d) Type BLV

Fig. 2 Configurations of box-type laterite masonry structures with roof

The addition of containment reinforcement or RC band increases the stiffness of the structure and hence it results in an increase in the natural frequencies of the structure. This was observed both in the case of structures without roof and those with roof. The fundamental mode shape in all the structures without roof (Types A, AV, ALR and ALRV) is the 'breathing mode', the opposite walls showing out-of-phase motion with maximum amplitude at the top edge of the walls as shown in Fig. 3(a). The fundamental mode shape of structures with roof (Types B, BV, BL and BLV), however, is the 'sway mode', the long walls showing in-phase motion as shown in Fig. 3(b).

Thus provision of an RC band or containment reinforcement alone does not seem to change the fundamental mode shape of box-type laterite masonry structures. But the provision of a heavy RC



(a) Without roof -Type A- Breathing mode (b) With roof - Type B – Sway mode

Fig. 3 Fundamental mode shape of box-type laterite masonry structures

roof slab seems to change the fundamental mode shape of single storeyed box type laterite masonry structures from a ‘breathing mode’ to a ‘sway mode’.

4. Equivalent-static analysis

Before attempting detailed time-history analyses, equivalent-static analyses were conducted in order to identify the most vulnerable regions of the laterite masonry structures, undergoing seismic movements. The design horizontal seismic coefficient for masonry structure, calculated according to IS 1893 (Part 1) – 2002, was obtained as 0.45. Hence, a horizontal acceleration of 0.5g, perpendicular to long walls, was applied on the entire height of the building.

The structural response of the buildings is then evaluated in terms of relative magnitudes of out-of-plane displacements of the long walls and the stresses in the masonry walls in horizontal and vertical directions.

4.1 Out of plane deflection (u_z) of long walls

Figs. 4 and 5 show the displacement contours perpendicular to long wall (u_z) for buildings of Type A and Type B respectively. The out of plane deflection patterns are similar in all the box-type laterite masonry structures without roof, (A, AV, ALR and ALRV), the maximum deflection being at the center of top edge. The out-of-plane deflection patterns are similar in all the box-type laterite masonry structures with roof, B, BV, BL and BLV. Figures 6 and 7 show the displacement profiles perpendicular to long wall (u_z) at mid length of the long wall, along the height, for buildings without roof and with roof, respectively.

The reduction in displacement u_z, in case of ALR and ALRV clearly shows the combined effect of lintel band and roof band. In box type laterite masonry structures without roof, simultaneous provision of both a lintel band and a roof band has proved to be more effective in reducing the displacement, than the vertical containment reinforcement. In structures with roof (B, BV, BL & BLV) the maximum deflection is observed near ¾ th height, along the center of the long wall. In

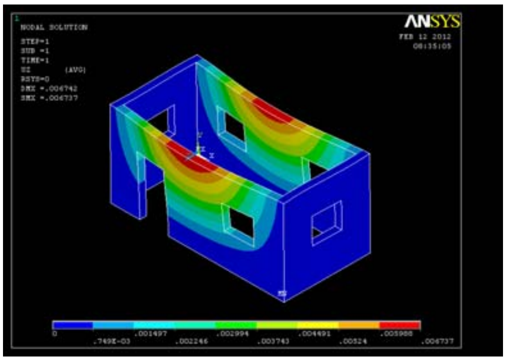


Fig. 4 Deflection (u_z) in building Type A

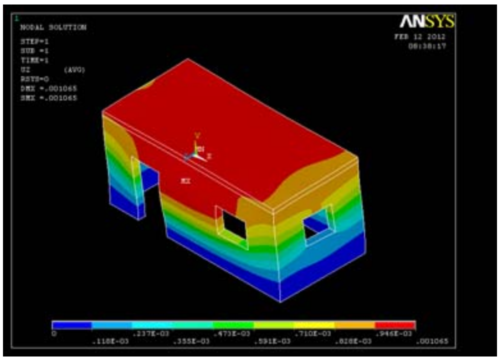


Fig. 5 Deflection (u_z) in building Type B

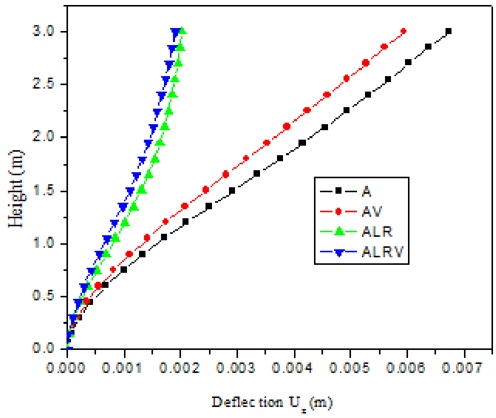


Fig. 6 Deflection (u_z) of long wall at mid length in structures without roof

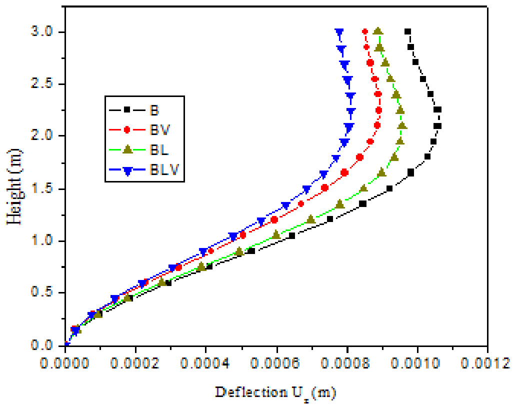


Fig. 7 Deflection (u_z) of long wall at mid length in structures with roof

box-type laterite masonry structures with roof, the rigid roof itself has helped to reduce the out-of-plane deflection u_z appreciably. RC bands and containment reinforcement have helped to reduce the deflection further. The effect of vertical containment reinforcement is relatively more in structures with roof, compared to structures without roof.

4.2 Bending stress σ_y perpendicular to bed joints in long walls

Fig. 8 shows the variation of stresses σ_y along the height of the long wall at the mid-length, for box-type laterite masonry structures without roof. The maximum value of 0.58 MPa for σ_y was observed near the center of base of the long wall, in building Type A. This was reduced to 0.48 MPa in building Type AV, a reduction of 17%. The maximum value of σ_y in building Types ALR and ALRV were noted as 0.34 MPa and 0.275 MPa, a reduction of 41% and 53% respectively, from that of building Type A. In box type laterite masonry structures without roof, RC bands have helped to reduce the stresses σ_y considerably. Containment reinforcement helped in reducing the stresses σ_y further. Hence in box type laterite masonry structures without roof provision of RC bands along with containment reinforcement helps to reduce the vertical stresses σ_y.

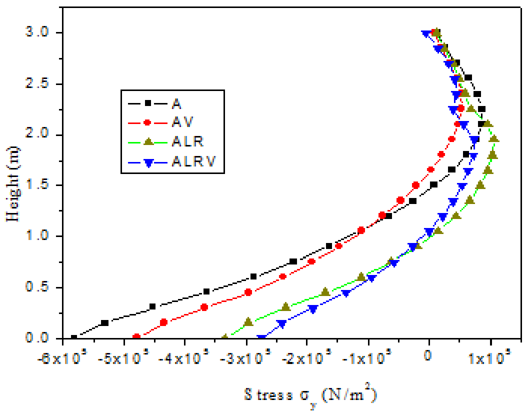


Fig. 8 Stresses σ_y near mid length of long wall along height in buildings without roof

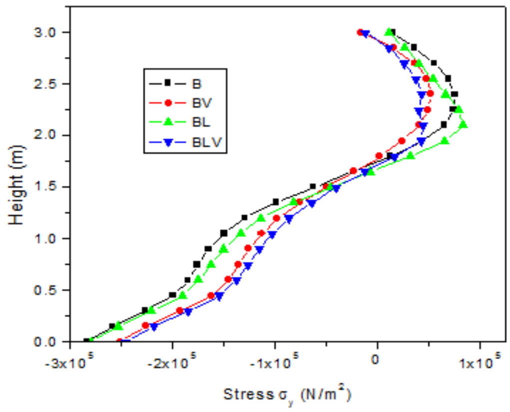


Fig. 9 Stresses σ_y at a corner of long wall along height in buildings with roof

The variation of stresses σ_y along the height at a corner of the long wall of box-type laterite masonry structures with roof is shown in Fig. 9. The maximum value of σ_y was observed near the corner of base of the long wall, 0.283 MPa in building Type B. This was reduced to 0.251 MPa in building Type BV, a reduction of 11%. The maximum value of σ_y in building Type BL was noted as 0.279 MPa which is nearly the same as the maximum value in building Type B. In BLV, the maximum value was 0.245 MPa, a reduction of 13% from that of building Type B. In box type laterite masonry structures with roof, the maximum value of stress σ_y is observed to be only about 50% of that of structures without roof. Again in these structures with roof, vertical containment reinforcement has played a better role than lintel band in reducing the stresses σ_y.

4.3 Bending stress σ_x parallel to bed joints in long walls

Fig. 10 shows the stresses σ_x parallel to bed joints in long walls of box type laterite masonry structures without roof. The maximum value of σ_x was observed around the center near the top edge of the long wall, 0.315 MPa in building Type A. This got reduced to 0.27 MPa in building Type AV, a reduction of 14%. The maximum value of σ_x in building Type ALR and ALRV were noted as 0.09 MPa and 0.084 MPa respectively, which are 71% and 73% smaller than the maximum σ_x in building type A. Thus in box-type laterite masonry structures without roof, RC bands have helped to reduce the stresses σ_x considerably and the effect of containment reinforcement is not much appreciable.

Fig. 11 shows the variation of stresses σ_x parallel to bed joints at sill level, where the stresses were observed to be maximum, in long walls of structures with roof (B, BV, BL and BLV). The maximum stress in laterite masonry structure B was noted as 0.038 MPa. This was reduced by 10%, 33% and 41% in structures of Types BL, BV and BLV respectively. The maximum value of stress σ_x in box type laterite masonry structure with roof i.e. Type B, is almost 88% less than that in structure without roof (i.e. Type A). In these structures with roof, however, containment reinforcement has proved to be more effective than providing a lintel band in reducing the stresses σ_x.

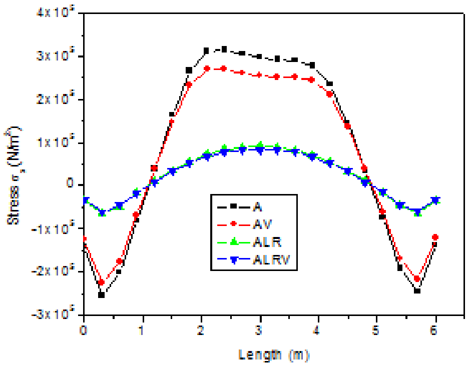


Fig. 10 Stresses σ_x near top edge of long wall in structures without roof

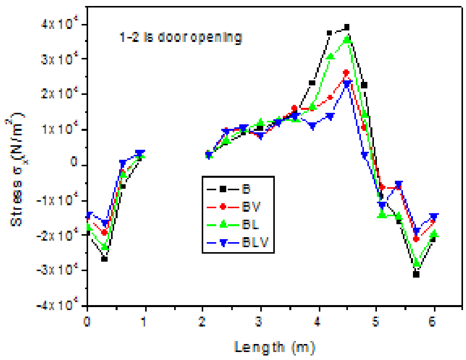


Fig. 11 Stresses σ_x at sill level of long wall in buildings with roof

5. Response of box type laterite masonry structures to El-Centro earthquake accelerations

The best way to evaluate seismic performance of structures is by characterizing their behavior under real earthquake records. A detailed time-history analysis can be conducted to calculate the response of a structure to any earthquake (Sahin 2010). In this study, a linear transient analysis was carried out with El-Centro (N-S component) earthquake acceleration record as an input. The north-south component of the ground motion acceleration recorded at a site in El-Centro during California earthquake of May 18, 1940 is shown in Fig. 12. Fig. 13 shows the Fourier amplitude spectrum of El-Centro NS component. The predominant frequency content of El-Centro acceleration record is around 1.17 Hz, which is significantly lower than the fundamental frequency of the buildings analyzed.

Although ANSYS may be used to implement a step-by-step time history analysis, it is not so easy to achieve the same using its graphical user interface, as a particular tool for seismic analysis is not available. There is no facility to apply a given acceleration time-history directly as base-node excitation, in Version 10 of ANSYS. Sahin (2010) developed an assistant program, named ANSeismic, for earthquake analyses of structures with ANSYS. He proposed application of the acceleration time-histories to the whole model with base fixed using the ACEL command available in ANSYS. In their technical report, the researchers from Korea institute of nuclear safety have established the efficacy of this method (Jhung 2009).

The following steps based upon the methods proposed by Sahin were adopted in this study: The database file of the model prepared in ANSYS is saved in a folder. A program written to do the transient analysis is saved as a text file in the same folder. A seismic record text file is also created and the downloaded acceleration records multiplied by gravity load is saved in this file which is assessed by the program. The damping value and earthquake direction are given in the program. First, modal analysis is executed and damping coefficients α and β are calculated using the first fundamental frequency assuming a damping ratio of 5 %. Then, the time history analysis is executed by applying each acceleration value to the model step by step in time domain.

Eight types of box type laterite masonry structures as detailed earlier were analyzed to find the response of these structures to El-Centro (N-S component) ground acceleration. The long cross

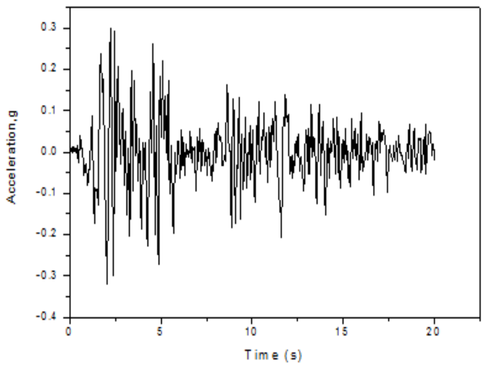


Fig.12 Acceleration time history of El-Centro (NS component)

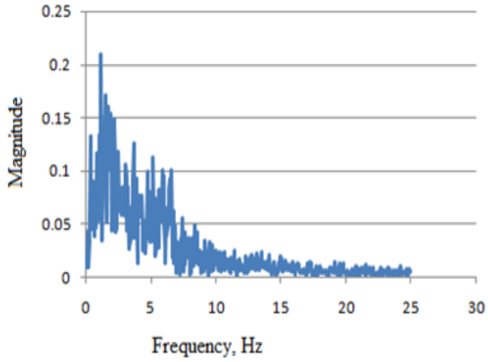


Fig.13 Fourier amplitude spectrum of El-Centro (NS component)

walls of the box-type laterite masonry structures are the ones that are vulnerable to out-of-plane flexural cracking. Hence the ground acceleration was applied perpendicular to the long walls of the box-type laterite masonry structures.

5.1 Time-history response of deflection u_z

Equivalent static analysis conducted earlier had revealed that the node at the center of top edge of the long wall recorded the maximum out-of-plane deflection u_z, for all the cases without roof and hence the time-history responses of only that node were plotted. Fig. 14 shows the time-history responses u_z of the node at center of top edge of long wall in buildings without roof.

The peak deflection in structure Type A was noted as 8.64 mm. This was observed to have reduced to 7.12 mm in structure AV, which is 18% less than that of structure Type A. The peak deflections in structure Type ALR and ALRV were similarly noted as 2.62 mm and 2.31 mm respectively, which are 70% and 73% less than that of structure Type A. The results show that in box type laterite masonry structures without roof, RC bands have reduced the out of plane deflections significantly.

Results of equivalent static analysis on box type laterite masonry structures with roof (B, BV, BL and BLV), indicated that the out-of-plane deflection of these structures were quite small as compared to those without roof. Nodes with peak deflections were identified from equivalent static analysis and the time history responses of those nodes under El-Centro earthquake were taken. Fig. 15 shows the time history responses of deflection u_z of the most vulnerable node, at 2.25 m height, along the center of the long wall in structures of Type B.

The peak deflection in structure Type B was noted as 1.1 mm. This was observed to have reduced to 0.93 mm in structure BV, which is 16% less than that of structure Type B. The corresponding peak deflections were noted as 0.995 mm and 0.85 mm respectively, in structure Type BL and BLV, which are 10% and 23% less than that of structure Type B.

The results show that the rigid roof itself has helped to reduce the out-of-plane deflections significantly. Containment reinforcement has helped to reduce the deflection further. While the lintel band alone has not helped to reduce the out-of-plane deflection significantly, but provision of

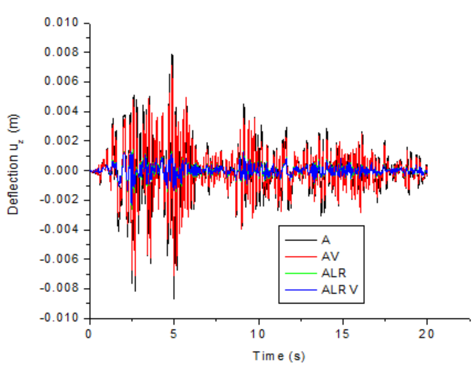


Fig. 14 Comparison of time history responses of deflection u_z (m) of the node at the center of top edge of long wall in buildings without roof

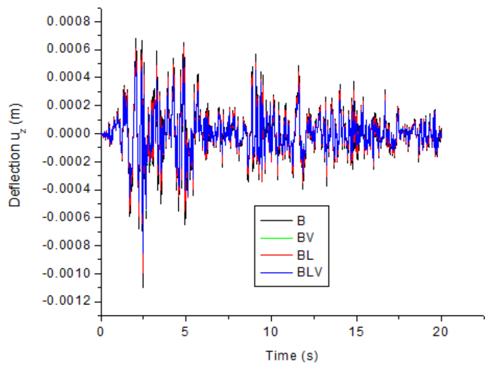


Fig. 15 Comparison of time history response of deflection u_z (m) of the node at center of long wall at ¾ th height of buildings with roof

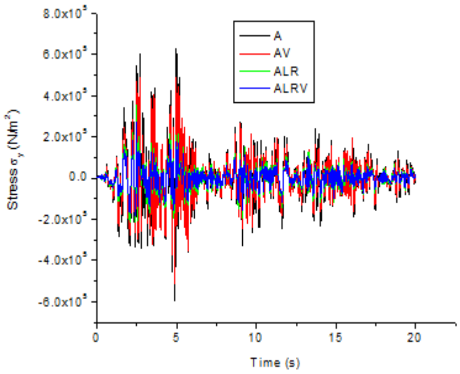


Fig. 16 Comparison of time history response of stress σ_y (N/m^2) of the node near the center of bottom edge in structures without roof

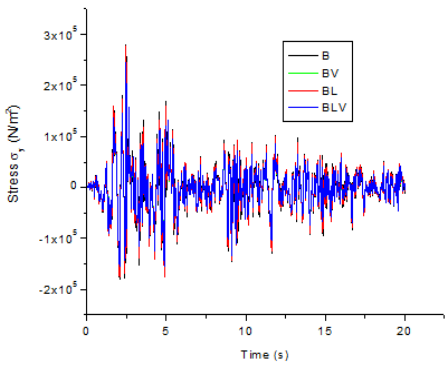


Fig. 17 Comparison of time history response of stress σ_y (N/m^2) of the node near the corner of bottom edge of long wall in structures with roof

lintel band employed along with containment reinforcement has helped to reduce the out-of-plane deflection by 23%.

5.2 Time-history response of stress σ_y in long walls

Nodes with maximum stress σ_y had been identified from equivalent static analysis and time history responses of stresses σ_y at those nodes were plotted. Fig. 16 compares the time history responses of vertical stress σ_y at the most vulnerable node for buildings without roof.

The maximum value of σ_y was observed as 0.63 MPa in building Type A. This has reduced to 0.49 MPa in building Type AV, a reduction of 22%. The maximum values of σ_y in building Type ALR and ALRV were noted as 0.35 MPa and 0.27 MPa respectively, reductions of 44% and 57% respectively, from that of building Type A.

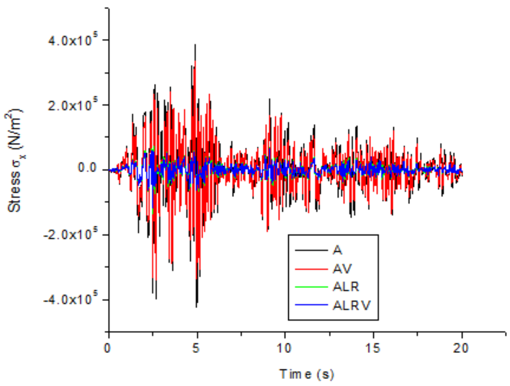


Fig. 18 Comparison of time history response of stress σ_x (N/m^2) of the most vulnerable node in structures without roof

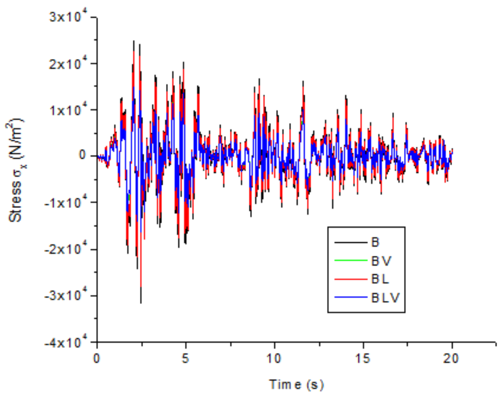


Fig. 19 Comparison of time history response of stress σ_x (N/m^2) of the most vulnerable node in structures with roof

In box-type laterite masonry structures without roof, RC bands have helped to reduce the stresses σ_y considerably. Still vertical containment reinforcement has helped to reduce the stresses σ_y further. Thus in box-type laterite masonry structures without roof, provision of RC bands along with containment reinforcement helps to reduce the stresses σ_y and thus reduce the chances of horizontal cracks in the masonry.

In box-type laterite masonry structures with roof, maximum stress σ_y was observed at the corner of base of long wall. Fig. 17 shows a comparison of the time history responses of the node near the corner of base edge, for stresses σ_y for buildings with roof.

The maximum value of σ_y was observed to be, 0.28 MPa in building Type B. This just reduced to 0.25 MPa in building Type BV, a reduction of 11%. The maximum value of σ_y in building Type BL was noted as 0.28 MPa which was the same as the maximum value in building Type B.

In case of BLV, the maximum value was 0.24 MPa a reduction of 14% from that of building Type B. Thus in box-type laterite masonry structures with roof, provision of the containment reinforcement may prove structurally more efficient than a lintel band in reducing the stress σ_y. In box-type laterite masonry structure with roof (B), the maximum value of stress σ_y was observed to be almost 55% less than that of structure without roof (A).

5.3 Time-history response of stress σ_x in long walls

For box type laterite masonry structures without roof, the maximum value of σ_x was observed around the center near the top edge of the long wall. The maximum value of σ_x was observed as 0.42 MPa in building Type A. This has reduced to 0.34 MPa in building Type AV, a reduction of 19%. Correspondingly the maximum values of σ_x in building Type ALR and ALRV were noted as 0.13 MPa and 0.12 MPa respectively, which are 69% and 71% less than the maximum σ_x in building Type A. Thus in box type laterite masonry structures without roof, RC bands have helped to reduce the stresses σ_x considerably and the effect of vertical containment reinforcement is not appreciable. A comparison of time-history responses σ_x at the most vulnerable node, as obtained from equivalent static analysis, in buildings without roof is presented in Fig. 18.

Fig. 19 shows a comparison of time-history responses of horizontal stress σ_x in structures with roof, for the most vulnerable node at the sill level, as obtained from equivalent static analysis.

The maximum value for stress σ_x in laterite masonry structure B was noted as 0.032 MPa. This got reduced to 0.028 MPa, 0.02 MPa and 0.016 MPa in structure Types BL, BV and BLV respectively, which are 13%, 38% and 50% lesser than that of structure Type B. The maximum value of stress σ_x in box type laterite masonry structure with roof (B) is almost 92% less than that in structure without roof (A). In these structures with roof, however, containment reinforcement has proved to be more effective than providing a lintel band in reducing the stresses σ_x. Such a containment reinforcement together with lintel band has helped to reduce the maximum stress σ_x by almost 50%.

6. Conclusions

The major conclusions drawn from the present investigation are as follows:

1. The fundamental mode shape of box-type laterite masonry structures without roof is the ‘breathing mode’, and that of such structures with roof is the ‘sway mode’.
2. Equivalent-static analyses conducted by applying a horizontal acceleration of 0.5g, perpendicular to the long wall, revealed that the long walls of box type laterite masonry structures without roof undergo a cantilever mode of deflection, with maximum deflection (u_z) at the center of top edge. Again in box type laterite masonry structures without roof, lintel band and roof band seems to be more effective in reducing the lateral displacement (u_z), than the containment reinforcement.
3. In a similar study on box type laterite masonry structures with roof, the maximum deflection was observed at about ¾th height, along the center of the long wall in all the four cases (B, BV, BL and BLV). The deflections seem to increase up to 2.25m height and then gradually decrease up to the top edge. The effect of containment reinforcement in reducing the out of plane deflection (u_z) is also observed to be slightly more in structures with roof, compared to structures without roof.
4. From equivalent static analyses, the maximum stress normal to bed joint σ_y was observed to occur near the center of the base of the long wall for building Types A, AV, ALR and ALRV. Although containment reinforcement helps to reduce the bending stresses σ_y in box type laterite masonry structures without roof, provision of RC bands appears to be more effective in reducing the stresses. Maximum stress σ_y was observed at the corners of base of the long wall for building Types B, BV, BL and BLV. In structures with roof, provision of containment reinforcement appears to be more effective in reducing the stresses σ_y than provision of a lintel band.
5. Equivalent static analyses reveal that the maximum values of stress σ_x, parallel to bed joints, occur at the center of long walls near the top edge for box type laterite masonry structures without roof (A, AV, ALR and ALRV). In box type laterite masonry structures with roof B, BV, BL and BLV, maximum stress σ_x was observed at sill level of window opening in the long walls. In these structures, providing containment reinforcement proves to be more effective than lintel bands in reducing the stresses σ_x.
6. Trends in reduction of out-of-plane deflection and bending stresses in transient analysis with earthquake accelerations match with those obtained from equivalent static analyses.

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