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PSEUDO-DYNAMIC EARTHQUAKE RESPONSE MODEL OF WOOD-FRAME WITH PLASTERED TYPHA (MINIMA) BALE MASONRY-INFILL

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ABSTRACT

This study investigated the design and evaluation of strength property of typha strawbale wall cross section. Assumption were made on edge column acting as axially loaded members that can resist vertical members from the loads acting on the wall. Based on this, the objective of this work is to provide the average design thickness for the cement-plastered typha strawbale that can stiffen the wooden frame. Data on strength and deformation of the structure are the input for the analytical models. Pseudo-dynamic earthquake response tests was conducted on one quarter (¼) scale model in a low rise storey wooden frames stiffened with cement plastered strawbale masonry. The structure was idealized as a plane frame. The analysis utilized the hysteresis models for members' models as time-independent. The force-displacement relationship of the members' models was evaluated by the approximate method on the basis of the material properties and structural geometry. The finite element model was *designed with straw bale infill panel to determine the hysteretic parameters, stiffness deterioration and strength degradation due to seismic forces.*

Key words: seismic; masonry-infill; typha strawbale, finite element; degradation, wood

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1. INTRODUCTION

A number of past studies focused on evaluating the experimental behavior of masonry infilled frames to obtain formulations of limit strength and equivalent stiffness (Mohammed and Amir 2012, Klinger and Bertero 1978; Bertero and Brokken 1983; Mander and Nair 1994). A more rigorous analysis of structures with masonry infilled frames requires an analytical model of the force – deformation response of masonry infill. While a number of finite element models have been developed to predict the response of infilled frames (Dhanasekar and Page 1986. Mosalam 1996, Malnotra 2002), such micro modeling is time-consuming for analysis of large structures. Alternatively, a macro model allows treatment of the entire infill panel as a single unit.

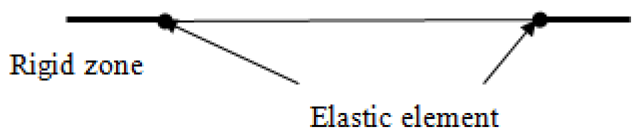
Saneinejad and Hobbs (1995) developed a method based on the equivalent diagonal strut approach for the analysis and design of frames with masonry infill walls subjected to in-plane forces. The model column that the shear walls acting independently of the ductile moment-resisting portions of the frame can resist the total required seismic forces. It is also required that design force distributions along elevations of the structure for both axial- flexural and shear design is the same. It was observed from the research carried out by Ahmet et al (1984) the universal basic column later claim could be realized by shear design based on calculation that demands that consider actual flexural capacity of the walls. The actual contribution of the edge members (columns and beams) panel RC/ prestressed concrete should be considered in evaluating available shear strength. Therefore the study has recommended that in designing and evaluating of strength of wall, cross section should be considered rather than the assumption of edge column acting as axially loaded members that should resist all vertical members from the loads acting on the wall.

The objective of this paper is to provide the average design thickness for the cement-plastered strawbale stiffened by the wooden frame. Data on strength and deformation of the structure are the input for the analytical models. The plastered strawbale wall was connected to the wooden frame with mortar (1:6 cement-sand mix) to constitute a simple support.

2. MODELLING OF STRUCTURAL MEMBERS

2.1. Frame: Beam and Column Members

A one-dimensional model was used for beam and column in this paper. The beam or column member was idealized as a perfectly idealized elastic mass-less line element with two nonlinear rotational springs at the two ends. The model as shown in fig. 1 have two rigid zones outside the rotational spring.



Non-linear rotational springs

Figure 1 One-dimensional model for beam and column

2.2. Analytical Model of Strawbale Infill

Shear walls can be idealized as (a) an equivalent column taking flexural and shear deformation into account, (b) a braced frame in which the shear deformation is represented by deformation of diagonal elements, where the structural deformation is by the deformation of vertical element and (c) short line represent along the height with each short segment with hysteretic characteristics. These models have advantages and disadvantages. In most cases the horizontal boundary beams are assumed to be rigid.

The proposed analytical model assumes that the contribution of the straw bale wall infill panel (Fig. 2a) to the response of the infilled frame can be modeled by replacing the panel by a system of two diagonal straw bale wall compression struts (Fig 2(b)). Since the tensile strength of masonry is negligible, the individual straw bale strut is considered to be in tension. However, the combination of both diagonal struts provides a lateral load resisting mechanism for the opposite lateral directions of loading. The lateral force-deformation relationship for the structural straw bale infill panel is assumed to be a smooth curve bounded by a bilinear strength envelope until the yield force V_y and then on a post yield degraded stiffness until the maximum force V_m is reached. The corresponding lateral displacement values are denoted as V_y and U_m respectively.

2.2.1. Strawbale wall as infill panel

The use of strawbale infill for the construction of a building in place of conventional materials is its cheapness, availability and flexibility in terms of workability and strength. Though the individual straw bale strut is considered to be in effective in tension, yet strawbale masonry has a high tensile strength. The load resisting mechanism of infill frames is idealized as a combination of moment resisting frame system formed by the frame and a pin-jointed system formed by the strawbale. However, because of the absence of a realistic, yet simple analytical model, the plastered straw bales as infill panels might be neglected in the non-linear analysis of building structures. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength, and ductility of the structure.

Saneinejad and Hobbs (1995) developed a method based on the equivalent diagonal strut approach for the analysis and design of steel or concrete frames with masonry infill walls subjected to in-plane forces. The method takes into account the elastoplastic behaviour of infilled frame considering the limited ductility of infill materials. The formulation provides only extreme or boundary values for design purposes. In the case of straw bale panel, the aspect ratio, shear stresses at the interface between the infill and frame, together with the frame strength were accounted for and the formulation expresses the boundary values for design purposes.

3. EQUIVALENT STRUT MODEL

The description, in brief, of the formulations for predicting the parameters of the strawbale infill-panel is presented in this section. Considering the straw-bale infilled frame as shown in Fig 2 (a) and (b), the maximum lateral force V_m and corresponding displacement U_m in the infill straw bale panel are expressed as

$$V_m^{+}(V_m^{-}) \le \frac{Vtl}{(1-0.45\tan\theta)\cos\theta} \le \frac{0.83tl}{\cos\theta} \qquad (1)$$

$$U_m^{+}(U_m^{-}) = \frac{\varepsilon_m L_d}{\cos\theta} \qquad (2)$$

where t = thickness of the infill panel: l' = lateral dimension of the wall panel : f_m = masonry strength, $\mathbf{s}_{\underline{m}}$ = corresponding strain, θ = inclination of the diagonal strut to the horizontal; $V = V_m \,\underline{\wedge}\, k$, V = shear strength (average of 0.347 N/mm^2), of straw bale wall; and A_d , L_d = area and length of the equivalent diagonal struts respectively calculated as,

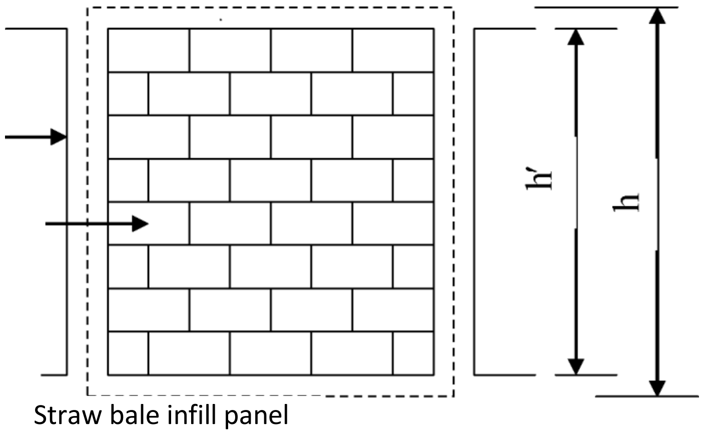


Figure 2 (a) Straw bale wall infill frame sub assemblage

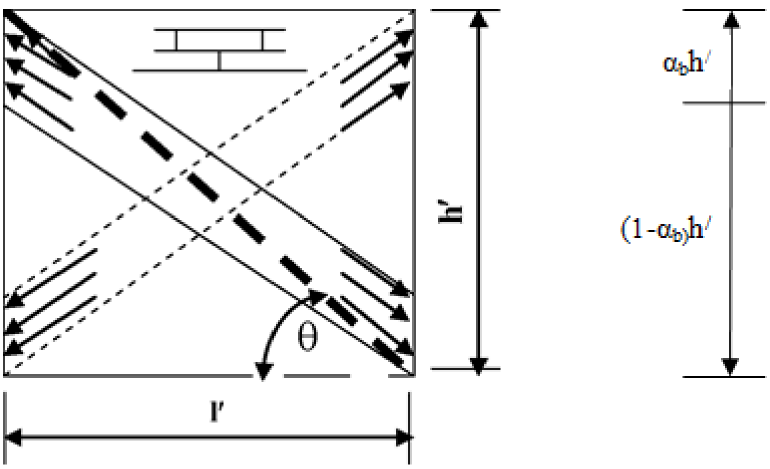


Figure 2 (b) Straw bale wall infill panel wall equivalent diagonal strut

4. SEISMIC LOAD ANALYSIS

4.1. Time-Independent Smooth Hysteresis Model

A smooth hysteretic model proposed by Beber and Wen (1981) is used for the structural straw bale masonry infill panel. The model, which was developed based on the Bouc-Wen model for hysteresis behaviour provides a smooth hysteresis force-displacement relationship between force V and displacement U given as,

$$V_i = V_y\,[a_m + (1 - a)\,Z_i] \qquad (3)$$

Where \x = ductility calculated as Ui/U_y, subscript i = instantaneous values, subscript y = yield values, and Z = hysteretic component determined by solving the following differential equation by Reinhorn (1995) as;

$$dZ_i = [a_{eff} \text{-}1\ Zi\ I^n\ [p\ \mathrm{sgn}\ (d_m\ Z) + y]]\ d\ \backslash i \qquad (4)$$

where signum function sgn(x) = 1 for x > 0 and = -1 for x < 0; a_{eff} = 3 , and y = constants that control the shape of the generated hysteretic loops (Assumed values a_{eff} = 1, and p = y = 0.5. At x = 0, the effect of cyclic loading is neglected and n controls the rate of transition from the elastic to yield state.

4.2. Stiffness Decay and Strength Degradation

The yielding system in general is the loss of stiffness due to deformation beyond yield point. The stiffness decay is incorporated directly in the hysteretic model by including the control parameter (r) in equation (4) for hysteretic parameter *Z* in which 77 is obtained by pivotal deterioration method (Valles et al, 1996).

Degrading systems such as straw bale wall infill panels will also exhibit loss of strength in the inelastic range. The strength deterioration is modeled by reducing the yield force V_y from the original value V_yo at each step k,

$$V_y^k = V_y^k\,(1 - DI) \qquad (5)$$

where DI = cumulative damage parameter dependent on the maximum attained ductility, |I max, and the cumulative energy dissipated (Valles et.al. 1996). Opening and closing of masonry cracks resulting in the pinching of hysteresis loops is a commonly observed phenomenon in masonry structural systems subjected to cyclic loading. The concept of slip lock element proposed by Baber and Noori (1984) was adopted in this study to formulate a hysteretic model.

5. EXPERIMENTAL TEST

5.1. Tested Structure and Properties of Materials

The tested structure is a three-storey one-bay (span ductile moment resisting wooden frame filled with typha strawbale masonry), built in a scale of one quarter (¼.) Analytical example was carried out for a full scale version of the test structure and all pertinent quantities were scaled using the length factor of 0.25. Each tested structure consists of strawbale wall (of 127 x 78 x 61 mm^3 unit size; typha straw bale plastered with 1: 6 cement-sand mix ratio, average density of 2,200 kN/m^3; 12% absorption and moisture content of 3.61. Its average compressive and flexural strengths were 2.83 and 1.56 N/mm^2 respectively whose principal plane was parallel with the direction of the input motion. Design gravity loads was 70 kN. Seismic design effects were determined using modal spectral analysis. The design spectral ordinates were set so that the first-mode shear base is equal to the design base shear required by the universal base column for a ductile moment-resisting space frame. Also, the response spectrum was selected to reflect soil condition that is superseded by a layer of sandy soil. Earthquake design actions were determined with a three dimensional elastic analysis model based on the cross- section of the columns and beams. The three dimensional structure was idealized as a pseudo-three-dimensional model in which only the planar modes of the main structural systems (walls and frames) were considered. Perfect base fixity of the frames was assumed. Horizontal floor level responses have been re-coded in lateral and main direction. At 3.2 m/s^2, the structure did not increase its response. The maximum base shear of 129 kN is obtained at the roof and a displacement of 12.3 mm.

The input signals to the shaking table modeled accelerated history for uniaxial tests are as indicated in Table 1 with a total of four earthquake records. These values were used as input ground motions in the dynamic analysis. These values and notations are used for individual ground motion, the components of earthquake motion record, peak ground acceleration (a_{eff}) in terms acceleration due to gravity (g), peak ground velocity (v_g), significant duration of the accelerogram (t_{SD}) and the normalized characteristics intensity (I_n).

Table 1 Characteristics of selected input earthquake ground motion

Ground motion	**Notation**	**Peak ground acceleration (a_{eff})**	**Peak ground velocity (v_g) m/s**	**Significant duration (t_{SD}), s**	**normalized characteristics intensity (I_n)**
El Centro	ELCENT	0.35g	0.34	24	2.66
Haruka-oki	HARU	0.42g	0.46	6	2.12
Silmar	SYLM	0.84g	0.61	47	1.71
Hacinohe	HACH	0.23g	0.34	28	1.52

6. A TYPICAL ANALYSIS EXAMPLE

6.1. Sections Properties

Second moment of area (I_b) = 2.278 x 10^{-3} m^4(I_c) = 6.75 x 10^{-4} mm^4 respectively. The straw bale wall is measured acting as diagonal members of the frame in this analysis. It acts as a stiffening element and the area and length of the equivalent diagonals at equivalent diagonal strut, L_d and its cross sectional area, A_d = 14235.0 mm.

6.2. Earthquake Analysis and Loading

For the absorption and dissipation of the energy through its motion, the frame of the building must be sufficiently ductile. For calculating ductility and other parameters it is assumed that the tests were entered to the ductile mode.

Equation (6) shows the calculated shear loading during seismic excitation. It is assumed that during a strong ground motion the inertial of a building results in a horizontal shear force at the base and proportional to the weight of the building and the imposed ground motion, so that:

$$V_D = C_s M_g \tag{6}$$

where V_D = total dynamic base shear, M = total mass of the building and its contents (kg), g = acceleration due to gravity, C_s = seismic coefficient, The lateral forces on the building are distributed over its height accordingly. The sum of the lateral forces equal to the total base shear. The lateral forces on the building frame are shown in Table 2.

Table 2 Seismic forces on Building frame

Storey level	Equivalent lateral force (kN)	Equivalent lateral shear force
Roof	41.45	41.45
2	71.65	113.10
1	143.3	256.40
Total dynamic base shear		256.4

The maximum lateral force and corresponding displacement in the infill straw bale panel are as follows:

At **9** = 320 at V_m^+ (V_m) = 14486.4N = 14486N, while the corresponding displacement U_m^+ (U_m) = 23.6mm. The initial stiffness K_o of the infill panel is estimated K_o =1227.63N/mm, the lateral yield force and displacement of the infill panel is 14193 .4 N and U_y $^+(U_y-)$ = 11. 7mm. Using different straw bale thickness, the result of the analysis for the stiffness deterioration and shear decay are not shown here for CASES II to IV. Maximun deflection of masonry wall is 24 mm. This value is greater than 23.7mm that was obtained in the analysis. Thus, the maximum deflection of the strawbale infill is less than the maximum allowed for masonry walls with the same dimensions (6.000 x 3.500 m). The results are validated with the measured responses for the base isolated morion (Table 3).

Table 3 Observed and measured response maxima for base isolated motion

Response	Table acceleration	Peak acceleration	Spectral acceleration	Base shear, kN	Total weight of the structure kN	Base moment kNm	Roof displacement Mm
Observed (prototype)	1.07	0.58	0.59	129	210	95.8	12.3
Calculated	-	0.1	-	143	365.62	158.6	23.6

6.3. Dynamic Response Results

At calculated Z =1, the stiffness decay is incorporated in the hysteretic model by including the control parameter ***T]*** for the hysteretic parameter (**r)**$_i$ > 1.0). The default value of S_k = 5 is recommended. The results in Table 4 shows a typical example of iteration of dynamic response characteristics from the analysis for 200mm and 250mm only.

Table 4 Dynamic response characteristics of straw bale infill thickness (t) = 200mm

Displacement (U_i)	Ductility (Hi)	Control parameter (rj_i)	Hysteretic Component	Lateral force (V_i), N	Shear decay ($V_m – V_i$), N
11.7	1.0	1	1	11042.5	112.70
11.8205	1.0103	0.9983	1.0017	11.062.22	92.98
11.9210	1.0189	0.9969	1.0031	11078.66	76.54
12.0045	1.0260	0.9957	1.0043	11092.32	62.88
12.0737	1.0319	0.9948	1.0053	11103.64	51.56
12.1311	1.0368	0.9940	1.0061	11113	42.20

Other dynamic response results for t = 275 and 300 are not shown here**,** but are represented in the graphical forms**.** The strength deterioration is modeled by reducing the yield force V_y from V_y^o at each step k.

7. RESULTS AND DISCUSSION

It was observed, from the result of the analysis, that the maximum lateral force of the the straw bale infill can be subjected to increase in the thickness of the infill panel. For thickness of 200mm,V_m = 11155.2N; for t = 250mm,V_m = 12071.9N; for t = 275mm, V_m =13279.2N and for t = 300mm, V_m = 14486N. Also, the stiffness and strength (hysteretic properties) of the straw bale wall infill decreased after the yield force. The yield force (V_y) being 14193.4N for t=300mm; 13145.1N for t = 275mm; 11949.9N for t = 250mm and 11042.5N for t = 200mm. The maximum deflection of the infill was obtained to be 23.7mm, which is less than the maximum 24mm specified for masonry infill walls. Also, it was found out that the hysteretic parameters deteriorate at a rate proportional to the thickness of the infill panel. As a result of the seismic loads, the force-displacement relationships for the loading cases, stiffness decay and the wall strength degradation are shown in Figs. (3) and (4) respectively. The hysteresis-energy dissipation due to seismic load was reduced detriment to damage of the structure.

8. CONCLUDING REMARKS

The use of straw bale wall as an infill panel for the proposed hysteretic model can compete effectively with other conventional materials, such as earth wall (Adedeji, 2002), since its maximum deflection U_m, is less than that specified for masonry walls. Though a perfect base fixity of the frames was assumed, the structure did not increase its response at 3.2 m/s^2. The maximum base shear of 129 kN is obtained at the roof displacement of 12.3 mm these are far less values than the analysis results.

The computed force-deformation response can be used to assess the overall structural damage and its distribution to a sufficient degree of accuracy. The structural materials characteristics were nominal values and may be different from the prescribed values. Such characteristics of materials for further analysis should be based on the confined and unconfined prestressed concrete. It was also observed that the structure shear decay increases with decrease in ductility of the wall infilled.

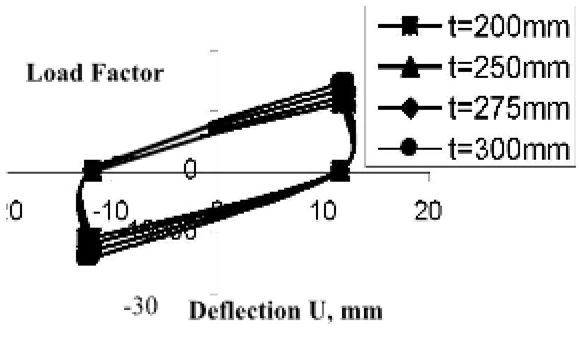


Fig. 3 Hysteresis curve with stiffness decay

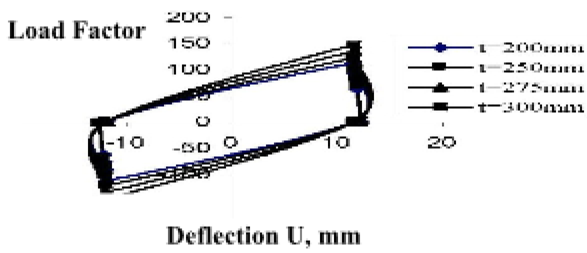


Figure 4 Hysteresis curve with strength degradation

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