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Design Approach for Load-Bearing Strawbale Walls

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Abstract

In recent years the use of stuccoed strawbales for wall construction on residential and light commercial has increased significantly. In load-bearing wall assembly applications the design engineer is faced with a structural system comprised of materials with dissimilar mechanical properties for which design parameters are not available in handbooks. This makes the design process challenging. In addition to presenting background information about load-bearing strawbale wall systems this paper presents the results of a series of tests conducted to gain more insight into the various parameters needed for the design of strawbale structures. These parameters include: dead load behaviour, bale response to over time, shear between straw and stucco, and axial load capacity of the stucco skin. Based on the test results the paper presents a design example for comparison of test values with design values.

1.0 Preamble:

Over the past three years various testing has been conducted on the use of rectangular straw bales for wall construction. The ongoing testing programme in the Biosystems Engineering Department at the University of Manitoba has investigated such things as: dead-load response of a bale wall, load-carrying capacity of stuccoed walls, shear resistance of the stucco/straw bond, and thermal performance. The purpose of this paper is to report on the structural performance of load-bearing stuccoed strawbale walls and to present a design approach that has been used on several structures in Manitoba and Ontario.

One of the biggest challenges for a design engineer working in the alternative buildings realm is with regards to what design values seem reasonable to use. Since there are no design handbooks for strawbale construction structural design of these buildings rests upon fundamental engineering principles and documented information of experience elsewhere throughout the world. While the vast majority of strawbale structures are being used for residences, their design falls outside of Part 9 of the National Building Code of Canada (NBCC, 95). Thus, in most cases, an engineer has to be involved in the design of these structures which requires a NBCC-95 , Part 4 analysis.

2.0 Introduction:

Before proceeding to specific design details it is important that the fundamental concepts behind load-bearing strawbale wall construction are discussed. Figure 1 illustrates a typical section through a strawbale wall assembly. For purposes of this report the wall has been broken down into three basic components:

i) Sill Assembly
ii) Wall
iii) Ladder Assembly.

2.1 Sill Assembly:

The sill assembly typically used for a slab-on-grade foundation is illustrated in Fig.2. It is comprised of nominal 38x89 (2x4) PWF nailing plates that are anchored to the concrete with 13mm (½") diameter wedge anchors spaced at 1200 mm (4'-0") on centre.

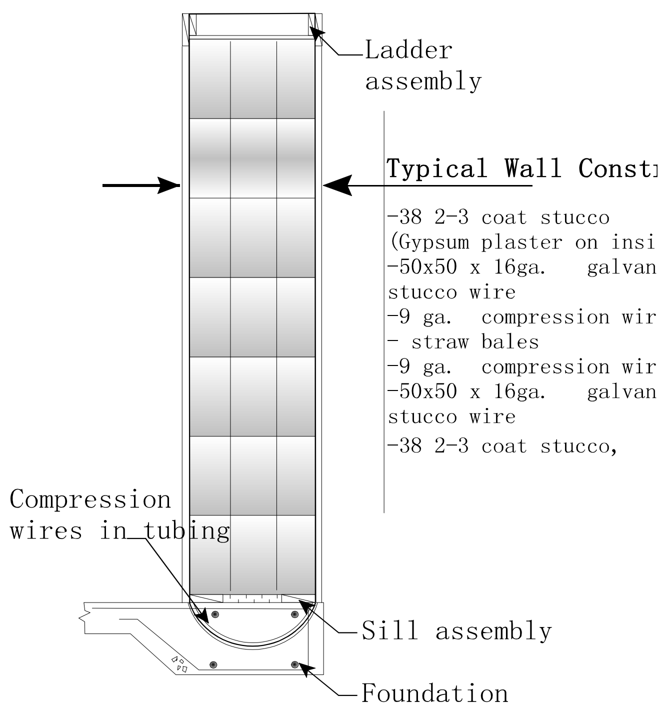

Fig.1: Section Through Strawbale Wall

Between the two nailers there is a layer of 38 mm (1-1/2") rigid insulation to reduce thermal bridging and also to provide a flat surface for the first course of bales. At the interface between the concrete and the nailer/insulation a layer of 6 mil CGSB vapour barrier is placed to retard any rising damp from the concrete foundation.

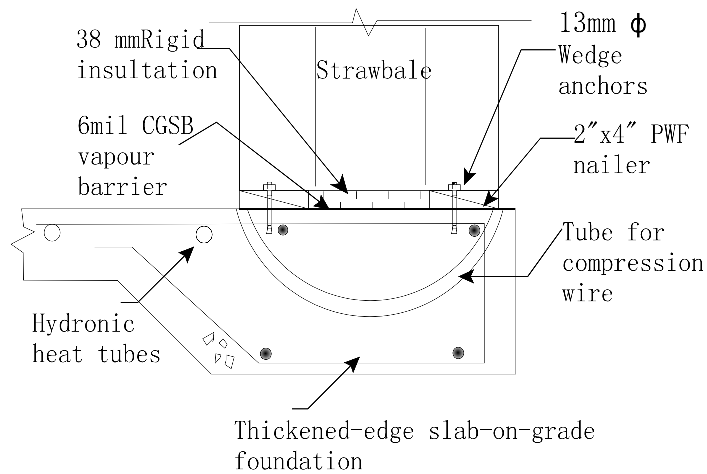

Fig.2: Sill Assembly

2.2 Wall Assembly:

As illustrated in Fig.1 the wall assembly is comprised of Portland-cement-based stucco applied to each side of a strawbale wall. The wall construction process is accomplished in four basic steps (Fig.4).

2.2.1 Stacking:

The straw bales used for these projects can come from a variety of sources such as: wheat, oats, barley and hemp. The bales are typically 460mm wide, 360 mm high by 900 mm in length (18 inches by 14-inches by 35-inches). To account for the natural variation in bales a density range is specified for construction. The bulk density of the bales is specified to be in the range of 5 - 7.5 pounds per cubic foot. The straw bales are stacked in a "running bond" pattern with all corners overlapped at alternating bale courses (Fig3). The straw bales are laid "on their strings" or flat on the 18-inch dimension. Walls are checked for plumb as the courses are laid. Once wall height is reached any further adjustment of the bales is done using a mallet to move them into place to achieve a vertical cross-section (Fig 4A).

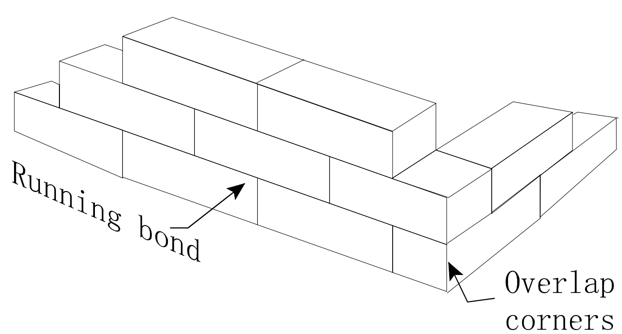

Fig.3: Bond Pattern and Corner

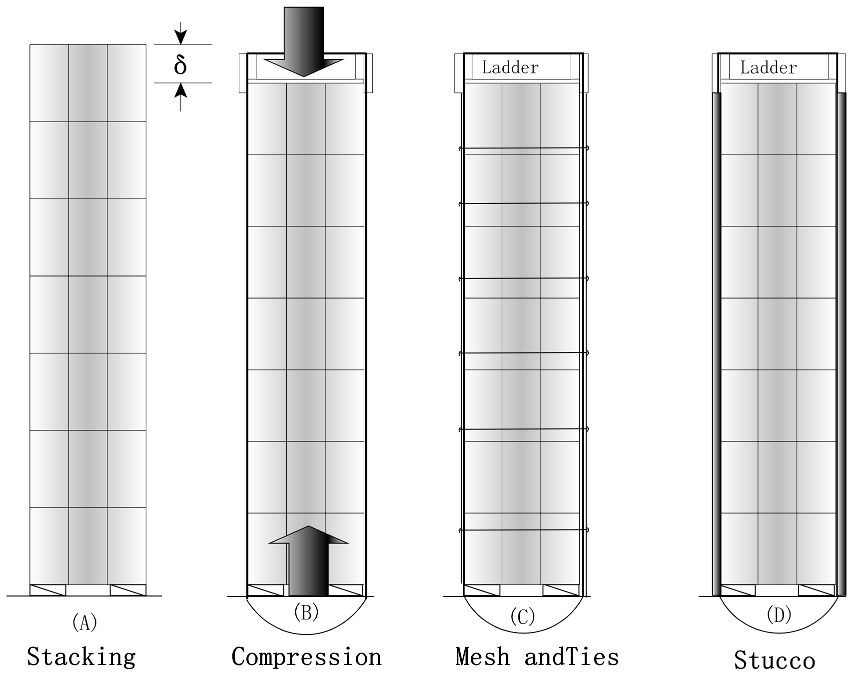

Fig.4: Four Basic Steps of Wall Construction

2.2.2 Compression:

Once the bale wall is constructed a ladder assembly is placed on top of the wall around the entire perimeter of the structure. The ladder assembly is fabricated from dimensional lumber and exterior grade plywood (Fig.5). The ladder fits over the top of the bale wall, with the outside (38 x 184mm) 2x8 dimensional lumber extending below the top of the bales as illustrated. A 9-gauge wire passes between the (38 x 140) 2x6 and (38x184) 2x8 on each side and is looped over the top. This wire encompasses the entire height of the bale wall. It is threaded through the tubing sleeve in the concrete (Fig.2), over the top of the ladder assembly and is overlapped on the

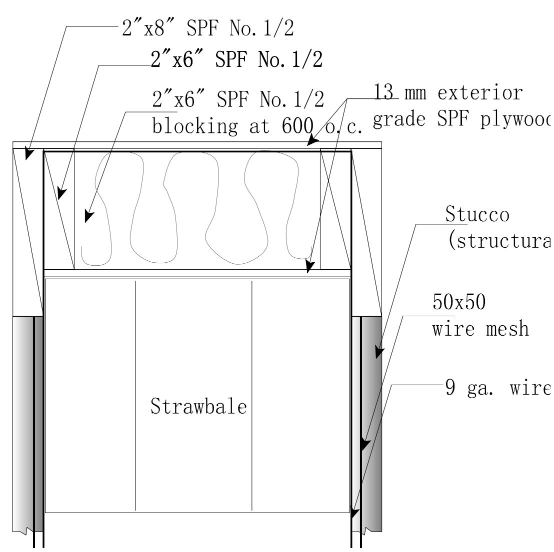

Fig.5: Ladder Assembly

inside of the wall where sufficient length is provided to grasp the wire with a fence stretcher. Once the wires have been installed at a spacing of not greater than 1200 mm on centre (4-foot), a fence stretcher is used to apply a tensile force to this loop. The result is the bale wall is compressed, deforming a vertical distance — * (Fig. 4B). This is typically referred to as the pre-compression phase of construction.

One of the first questions that emerged revolved around how long to leave the compressed wall prior to stucco application. It was felt that this time period was needed allow for settling of the bales and re-distribution of internal forces. Hands-on experience and "engineering judgement" seemed to indicate that a period of not less than 24 hours was required. To investigate this behaviour four series of tests were conducted to monitor the load carrying capacity of an un-stuccoed strawbale wall with time. A six-course bale wall approximately 2.75 metres (9-feet) in length was loaded uniformly along its length. At a maximum vertical deformation of 150 mm (6 inches) the loading was stopped and left on the test specimen for a period of not less than 72 hours. The plot contained in Fig. 6 illustrates the load versus time relationship that was determined (Arbour, 2000, Blum, 2002). It can be seen that after approximately 26 hours the load on the wall section became constant. While the tests were done using oat and wheat-straw only, it appears from these results that the initial recommendation to use a minimum of 24 hours would seem appropriate for design. The average value for the load-carrying capacity of the six-course oat straw bale wall at the point at which the curve becomes horizontal (C_{26}) was determined to be 5.8 kN/m (400 pounds per lineal foot).

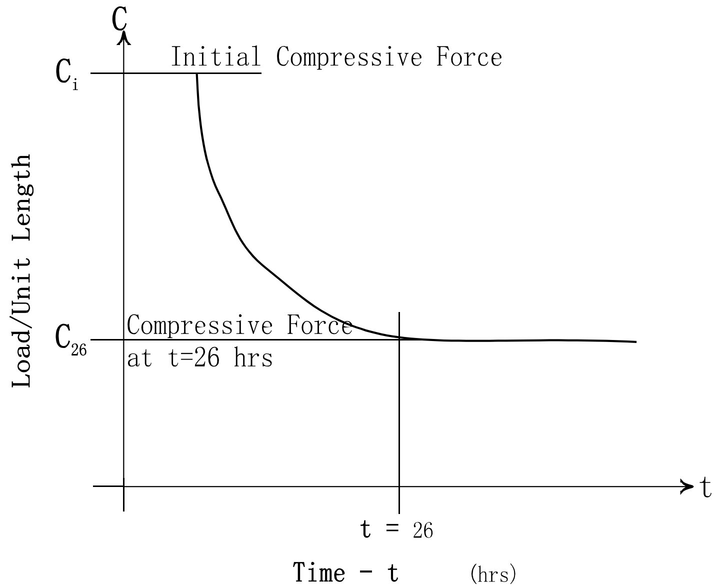

Fig. 6: Precompression vs Time
(Source: Arbour,2000; Blum, 2002)

What is considered to be equal importance is that the reduction in load-carrying capacity of the wall section does not occur as a result of further vertical deformation. This is important from a construction point-of-view in that once the wall has been compressed initially only minor adjustments are typically required to level the wall for the next phase of construction.

2.2.3 Ladder Assembly

The ladder assembly, described in the previous section, is placed on top of the wall. It provides a means to distribute pre-compression forces and structural loads on the straw-stucco wall system. The ladder used for the Seigel / Cochrane residence has a 2x6 framework that corresponds to the width of the bales and 2x8 dimensional lumber on each side to form a pocket (Fig5). These 2x8's are in direct contact with the stucco finish, providing a load path through the stucco and subsequently into the foundation. Figure 7 illustrates the concept behind a load-bearing strawbale wall. A pre-compression force “A” is applied to the wall prior to any stucco application (Fig.7A). The strawbales in the wall will continue to “see” this load, or, in other words, provide load-carrying capacity equivalent to the pre-compression force. Once the stucco process is complete, subsequent loading — “B”— can be applied. As load “B” is increased, the wall system does not react to it until load “B” approaches the magnitude of the pre-compression force - Load “A” (Fig. 7B). Once load “B” is in excess of “A” it is assumed to follow a load path through the stucco skin (Fig. 7C).

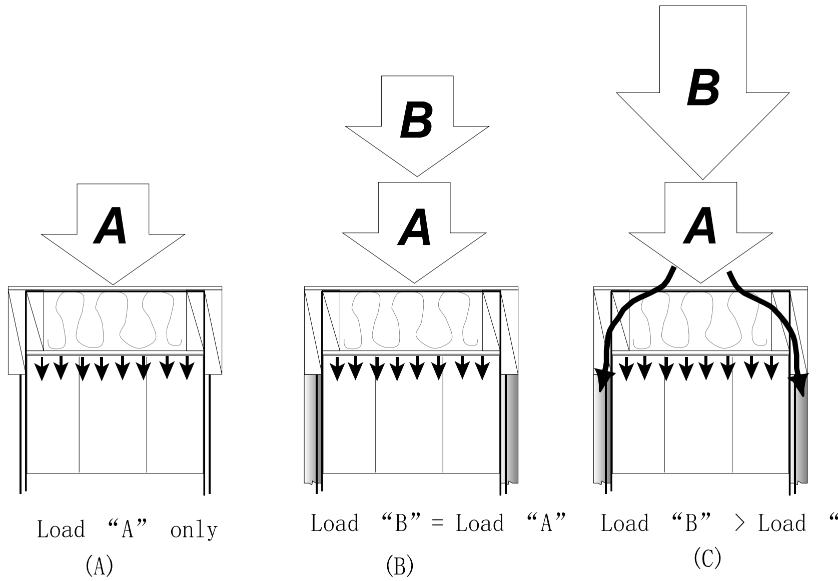

Fig.7: Load Transfer into Wall System

3.0 Design Process:

Three fundamental structural parameters are evaluated in the design of a load-bearing strawbale wall. The wall must provide resistance to: i) vertical load, ii) lateral load and, iii) racking resistance. The following outlines a design process that has been used on existing strawbale structures.

3.1 Vertical Load:

The wall must support gravity and live loads from the second floor and roof system. The load applied to the top of the wall through the ladder follows a load path through the stucco skin as described in the previous section and illustrated in Fig.8. A theoretical load capacity may be determined based on the following design assumptions:

i) Unreinforced, type N mortar with a compressive strength of 5.17MPa (750 psi) . A value of 2.06 MPa (300 psi) will be used for theoretical calculations.

ii) Stucco skins that are 38 mm (1-1/2") thick on each side. Reinforced with 16 ga. galvanized 50 mm x 50 mm (2" x 2") wire mesh on the inside face.

iii) The skins are laterally supported with ties that are fastened to the wire mesh on each side and pass through the strawbales. The ties are spaced at not greater than 400 mm (16") on centre each way linking the two skins together with straw sandwiched in between.

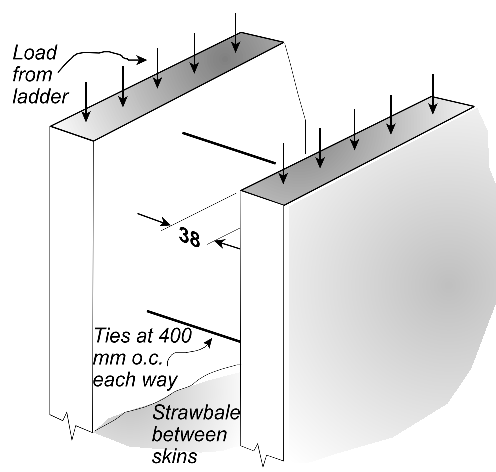

Fig. 8: Load Applied to Stucco Skin

Using the assumption of 2.06 MPa for the compressive strength of the mortar in conjunction with a 38 mm stucco skin on each side of the bale wall a theoretical strength may be determined as follows for one lineal metre of wall. For every metre of wall there is:

$$(38mm \times 1000mm) \times 2\ skins = 76{,}000\ mm^2 / m\ of\ wall \qquad (1)$$

Based on a 2.06 MPa compressive design strength, a theoretical load-carrying capacity may be determined as:

$$2.06\frac{N}{mm^2} \times 76{,}000\frac{mm^2}{m} = 156560\frac{N}{m} \quad (10{,}730\frac{lbs}{ft}) \qquad (2)$$

Figure 9 contains a plot of the ultimate vertical load values obtained from various test results and theoretical calculations. The values range from approximately 20 kN/m up to the theoretical value of 156 kN/m.. The average vertical load from tests conducted in the Biosystems Engineering Department at the University of Manitoba are indicated as number Ñ in Fig. 9. Two stuccoed wall sections were tested. Each section had six courses of bales with an overall length of approximately 2.75 metres (9 feet). Two coats of Portland-cement based stucco were applied to each side of the wall section. Load was applied uniformly along the length of the wall section. It should be noted that while these experimental values appear approximately at midrange, the test load was limited to the capacity of the test equipment. Both test wall sections sustained the maximum load of 33 kN/m (2250 plf) that could be provided by the test apparatus without any noticeable signs of distress. Further testing will be conducted with test apparatus with a higher capacity.

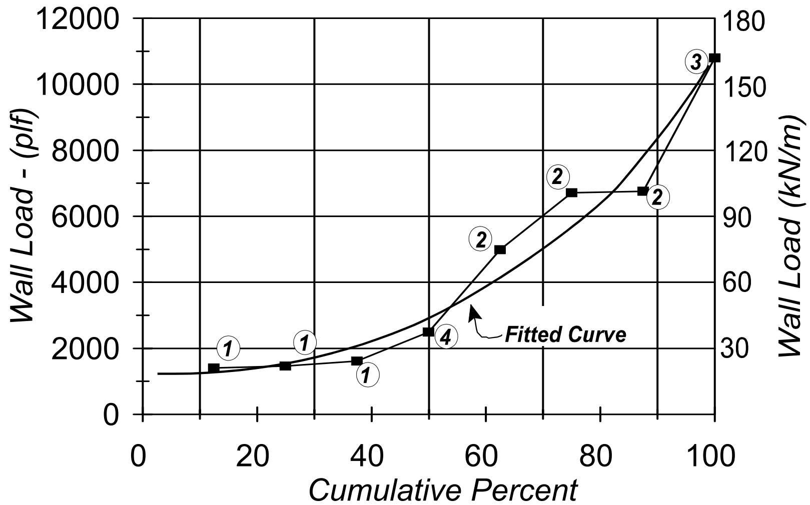

Fig.9: Ultimate Vertical Load -various sources
(Source: Î - Carrick,1998 Ï Grandsaert, 1999. Ð Theoretical Ñ Dreger, 2002)

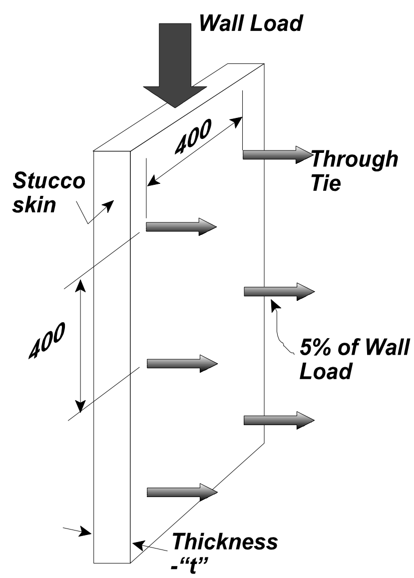

Fig.10. Lateral Resistance

Theoretically, the majority of the vertical load is carried by the stucco skins. These skins act as slender columns that get lateral support from the bond with the straw and the ties that run through the bales normal to the stucco skins. Lateral stability of the stucco skins is considered to be provided by the polypropylene ties that run through the bale wall, connecting the wire mesh on each side of the wall together. The net effect of these ties is to reduce the slenderness ratio to a point where slenderness effect will be considered minimal. As noted earlier, these ties are spaced at a maximum of 400 mm (16") on centre each way(Fig.10). If a typical vertical wall load for a residential structure is assumed to be 17.5 kN/m , and the compression of the bales maintains a nominal 4 kN/m then the net force in the stucco skins will be approximately 13.5 kN/m (925 plf). Based on a 400 mm tie spacing and a 5% lateral force[1] , the ties are required to provide 0.2 kN (45 lbs) resistance, which is considered to be provided with this system. Since the stucco skins act as a compression member this restraint provides for lateral stability of the wall system. With a tie-spacing of 400 mm on centre, the slenderness ratio is then determined to be:

$$Slenderness\ Ratio = \frac{l}{0.3h} = \frac{400mm}{(0.3 \times 38mm)} = 35.1 \qquad (4.)$$

where: h = thickness of the stucco skin

Based on slenderness criteria as prescribed in Clause 10.15.2 in CSA-A23.3-M94 slenderness effects are not considered to be critical to the overall performance of the wall system.

3.2 Lateral Load - Wind:

Wind acting on the structure require the wall system provide resistance to lateral loading. This loading can create a variety of conditions that the wall system must resist:

i) Flexural stress and deformation in a wall when the load is transverse to the wall face
ii) Shear stress in walls parallel to the direction of the wind.

3.2.1 Flexural Stress in Wall Panel:

Flexural stresses are created in the wall system as a result of the external and internal pressures as a result of wind acting on the structure (Fig.11). For purposes of illustration assume, based on Section 4.1.8 of the NBCC-1995, that the reference velocity pressure for a geographic location in Canada is given as: $q_{1/30}$ = 0.32 kPa (6.68 psf).

[1].5% of the wall load is considered a conservative value when compared to wood or steel design that typically uses 2% of the axial compressive force to represent the lateral bracing force.

The specified external pressure for the residence was determined to be[2]:

$$p = q \times C_e C_p C_g$$
$$p = 0.32 \times 0.9 \times 1.3 = 0.37\, kPa\ (7.8\, psf) \qquad (5)$$

The specified internal pressure was determined to be[3] :

$$p_i = q \times C_e \times C_g \times C_{pi}$$
$$p_i = 0.32 \times 0.9 \times 1.0 \times 0.7 = 0.2 kPa\ (4.2 psf) \qquad (6)$$

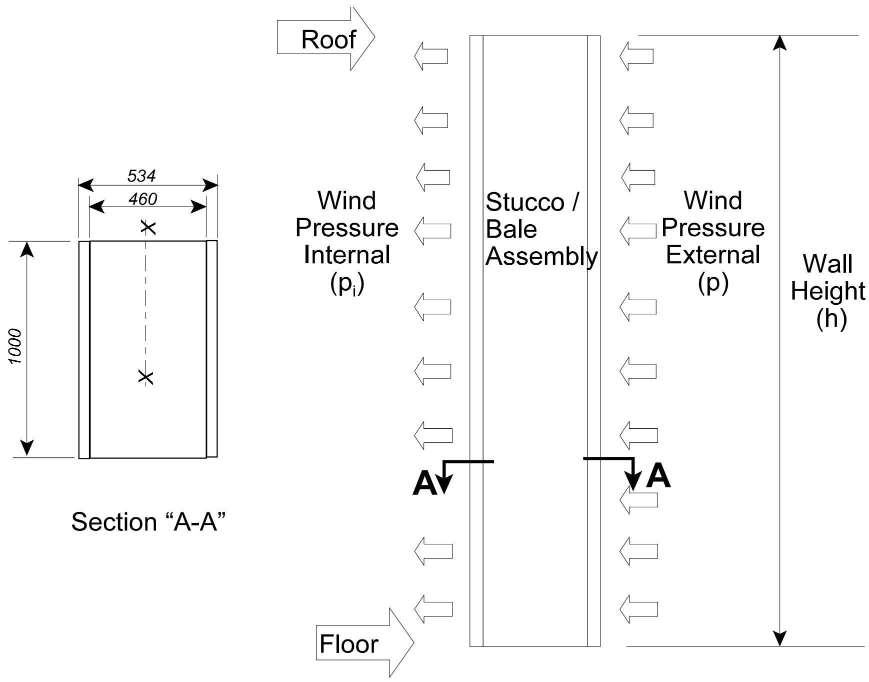

Fig.11: Lateral Wind Load on Wall System

Based on the above loading, the wall is subjected to a total of 0.57 kPa (12 psf). Using a one-metre strip of the wall, and a uniformly-distributed load of 0.57kN per metre the simply-supported bending moment in the wall can be calculated. In the determination of the flexural stress the moment of inertia of the wall assembly must be considered. It is apparent from both research and anecdotal evidence that there is a significant bond established between the stucco and the straw which could be taken into account for the purpose of moment of inertia determination. For the purposes of this design check, however, it was considered appropriate to use only the stucco skins in the moment of inertia calculation. Thus, using only the skins the moment of inertia for the section about the neutral axis was determined to be 4.58×10^9 mm^4.

Based on the following assumptions:

[2] The above relationship assumes the end section 1E with wind predominantly perpendicular to the ridge of the structure as per Fig.B7, NBCC supplement.

[3] Based on a Category 2 structure

- a wall height of 2.4 metres (8 feet),
- a live-load factor or "$_L$=1.5,
- neglecting the contribution of the straw to the moment of inertia,
- neglecting any pre-compression in the bales due to the assembly process and dead weight of the roof, and,
- a simply-supported moment, the flexural stress was calculated as:

$$\sigma_f = \frac{\alpha_L My}{I} = \frac{\left(1.5 \times \left(\frac{0.57 \times (2.4 \times 10^3)^2}{8}\right)\right) \times \frac{534}{2}}{4.58 \times 10^9} = 0.036\,MPa\ (5.2\,psi) \qquad (7)$$

The resulting flexural stress is considered to be minimal. If the maximum tensile stress in the mortar is limited to 5% of the compressive design stress, then the maximum specified stress (F_r) in tension would be 2.06 MPa x 0.05 = 0.1 MPa (15 psi.). In this example the ratio between the factored flexural stress F_f and the specified stress F_r represents on 36% of the assumed maximum specified stress. It appears that for typical wind-load situations flexural stress is not considered to be critical. When the above result is combined with the knowledge that there is additional compressive stress in the stucco skins from the roof dead load then flexural tension stresses will be of less concern.

3.2.2 Shear Resistance in Wall Panel

Shear forces may be present in a wall panel as a result of lateral forces applied to the face of a wall due to wind, and also from load transfer into end shear walls. The bond between the stuccoed skin and straw substrate in strawbale walls provide considerable shear resistance . Shear has been
investigated and results published based on both shear flow (force per unit length - kN/m, plf) and shear stress (force per unit area - MPa , psf). The following represent examples of typical values for these shear parameters[4]:

Shear Flow: - 10.94 kN/m (750 plf) (White and Iwanicha, 1997)
- 14.66 kN/m (1005 plf) (Boynton, 1999)
Shear Stress: - 2.0 kPa (42 psf) (Riley et al., 1998)

A series of tests were conducted in the Biosystems Engineering Department at the University of Manitoba to investigate the behaviour of the bond between Portland-cement based stucco and

[4] Results were originally published in Imperial units as indicated in brackets . Converted for this paper.

wheat straw bales (Stepnuk, L., 2002). A total of twelve single-bale specimens were tested. Each bale had two 38 mm stuccoed sides with 50mm x 50 mm (2"x2") galvanized stucco mesh.. Six of the specimens had polypropylene ties between the two sides that tied the mesh together. The other six specimens did not have through ties. Load from a universal testing machine was applied along the length of the stucco skins. The bale was supported on a platform that was slightly less than the width of the bale to allow the stuccoed skins to move. Load versus vertical deformation was recorded for each specimen. A t-test was performed on the data from the two batches - tied and untied bales. At a 95% confidence level there was no significant difference, thus the two batches were analysed as one data set.

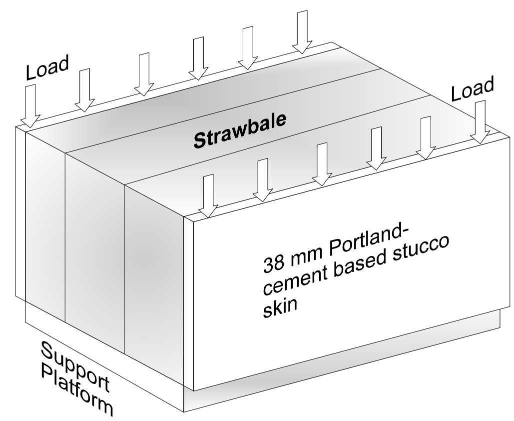

Fig.12: Shear Test Specimen

The ultimate shear-flow values determined from this investigation ranged from 5.04 to 18.53 kN/m. (345 to 1270 plf). These values are comparable with published results from other tests.

Within the context of the above design example the factored shear flow and shear stress can be determined for the 0.57 kPa loading.

$$Shear\ Flow = f = \frac{\alpha_L VQ}{I} = \left(\frac{1.5 \times 684N \times (8.74 \times 10^6 mm^3)}{4.58 \times 10^9 mm^4} \right) = 1.95 N/mm \quad (40.8 plf\ /\ ft\ width) \tag{8}$$

$$Shear\ Stress - \tau = \frac{\alpha_L VQ}{Ib} = \left(\frac{1.5 \times 684N \times (8.74 \times 10^6 mm^3)}{4.58 \times 10^9 mm^4 \times 1000mm} \right) = 0.00195 MPa \quad (0.28 psi) \tag{9}$$

Based on the design example for typical residential loading it appears that there is sufficient capacity in the wall system.

4.0 Recommendations for Further Study

Research on various aspects of load-bearing and non-load bearing strawbale wall systems is ongoing in the Biosystems Engineering Department at the University of Manitoba. The direction for ongoing research include such topics as:

- Shear testing of wall panels to determine racking resistance ;
- Testing to investigate force transfer paths along the stucco/strawbale interface;
- Moisture migration through bale wall systems in harsh environments;
- Load-carrying behaviour of earth-plastered wall systems;
- Thermal performance of bale walls with strawbale on edge;
- Application of hemp fibre reinforced stucco to replace wire mesh reinforcement, and;
- Long-term load behaviour of stuccoed wall systems .

5.0 Conclusion :

The use of rectangular strawbales in the construction of alternative wall systems has been shown to be a viable option for residential and light-commercial applications. This paper has outlined some aspects of design that can be applied to strawbale construction. Based on testing to date the load-carrying capacity of stuccoed strawbale wall systems with respect to vertical load, flexural capacity and shear performance appear to exceed minimum structural load requirements for many applications.

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