Report to the Ontario Building Code Commission
-Request for Further Information
Application 2001-52 , Seigel / Cochrane Residence
3715 Townline , RR#2, Orillia, Ontario

1.0 Preamble:

The purpose of this report is to respond to a request for further information regarding the Seigel / Cochrane strawbale house project in the Township of Severn , Ontario the Ontario Building Code Commission (BCC). Two areas of concern requiring further information were indicated in correspondence from the BCC regarding Hearing Number 01-46-839. These areas are:

i) moisture / air management issues and,

ii) the basis of design for the load-carrying capacity of load-bearing strawbale walls as designed for the Seigel / Cochrane residence.

The information presented in this report is limited to structural issues.

2.0 Introduction:

In its request for further information, the BCC outlined various options upon which this analysis could be based. The design process and parameters used for the Seigel / Cochrane residence is considered to be a combination of generally established theory and information obtained from full-scale testing. Since this structure falls under Part 4 of the Ontario Building Code - 1997 (OBC-97) analysis and design procedures will be linked to these requirements.

Before proceeding to specific design details it is important that the fundamental concepts behind the type of load-bearing strawbale wall construction used for this project be presented. Figure 1 illustrates a typical section through the strawbale wall used for the Seigel / Cochrane residence. For purposes of this report the wall has been broken down into three basic components:

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

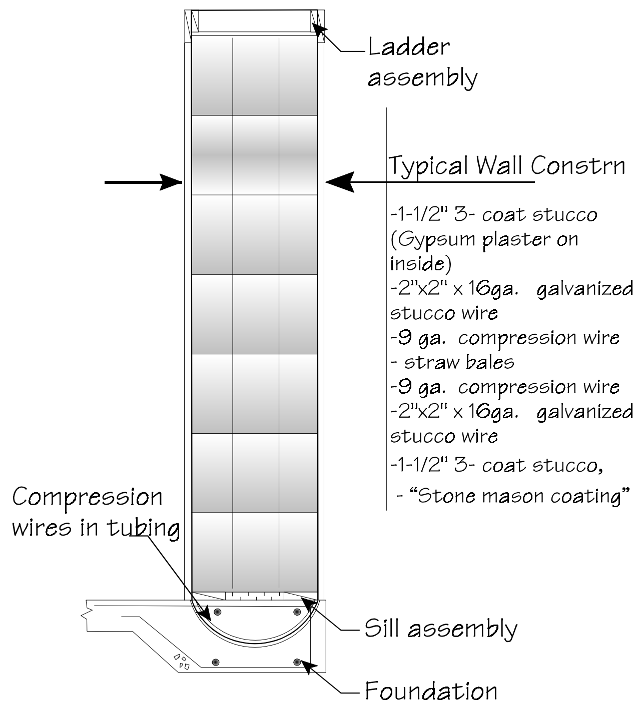

Fig.1: Section Through Strawbale Wall

2.1 Sill Assembly:

The sill assembly used for this project is illustrated in Fig.2. It is comprised of nominal 2"x4" PWF nailing plates that are anchored to the concrete with ½" diameter wedge anchors spaced at 4'-0" on centre. Between the two nailers there is a layer of 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.

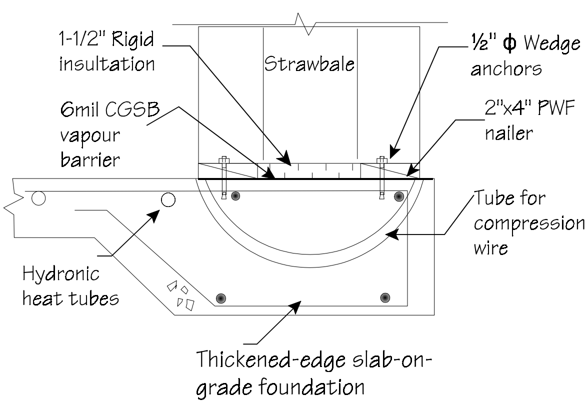

Fig.2: Sill Assembly

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:

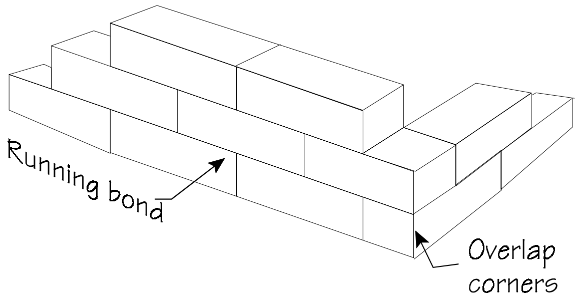

Fig.3: Bond Pattern and Corner

The straw bales used for this project are approximately 18 inches wide, 14-inches high by 35- inches in length. The type of straw used is oats. The bulk density of the bales is specified to be in the range of 6.5 - 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).

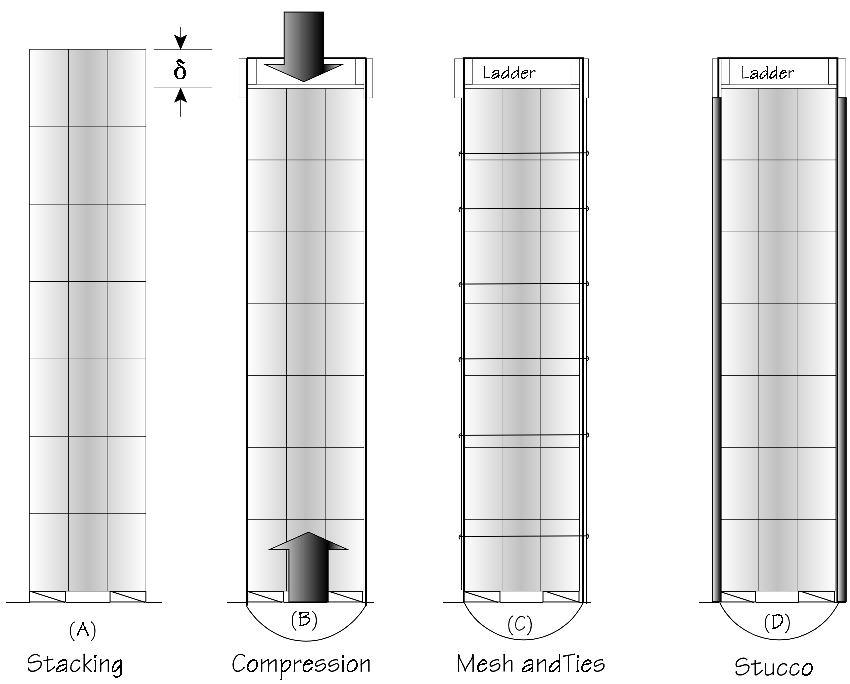

Fig.4: Four Basic Steps of Wall Construction

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 2x8 dimensional lumber extending below the top of the bales as illustrated. A 9-gauge wire passes between the 2x6 and 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 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 4'-0" on centre, 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. The specifications for this project indicate that once the initial compression phase is completed the wall must be allowed to settle for a minimum of 24 hours. This time-frame is based on research that indicates oat straw takes approximately 24-26 hours to redistribute the applied compression force (Arbour, 2000). The plot in Figure 6 illustrates this relationship. The research also indicates that this reduction does not occur as a result of further deformation. Thus, with oat straw within the specified density range a deformation criterion appears acceptable for design. The average value for the load-carrying capacity of the wall at the point at which the curve becomes horizontal (C_{26}) was 400 pounds per lineal foot.

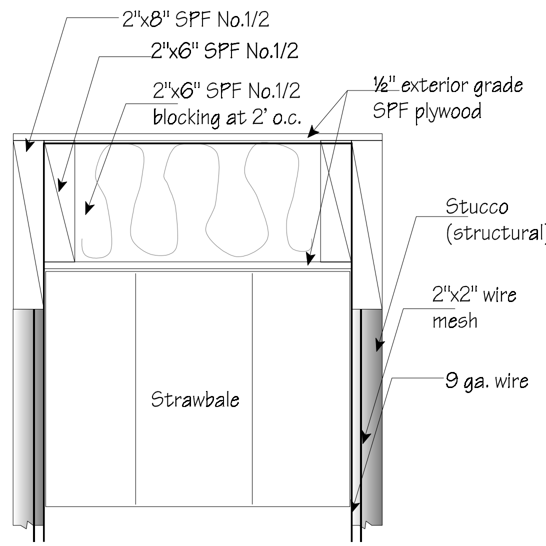

Fig.5: Ladder Assembly

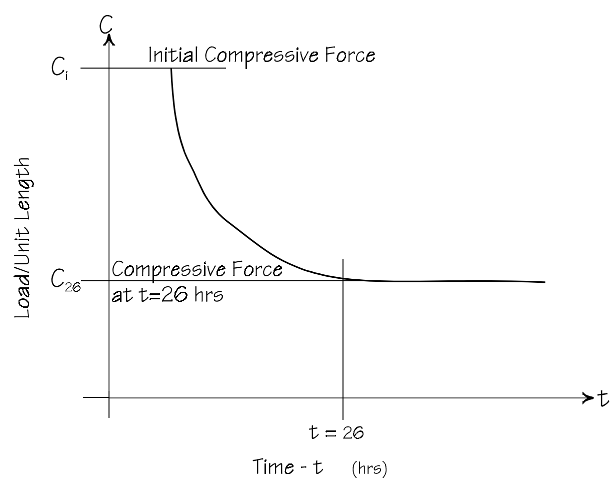

Fig. 6: Precompression vs Time - Oat Straw
(Source: Arbour,2000)

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).

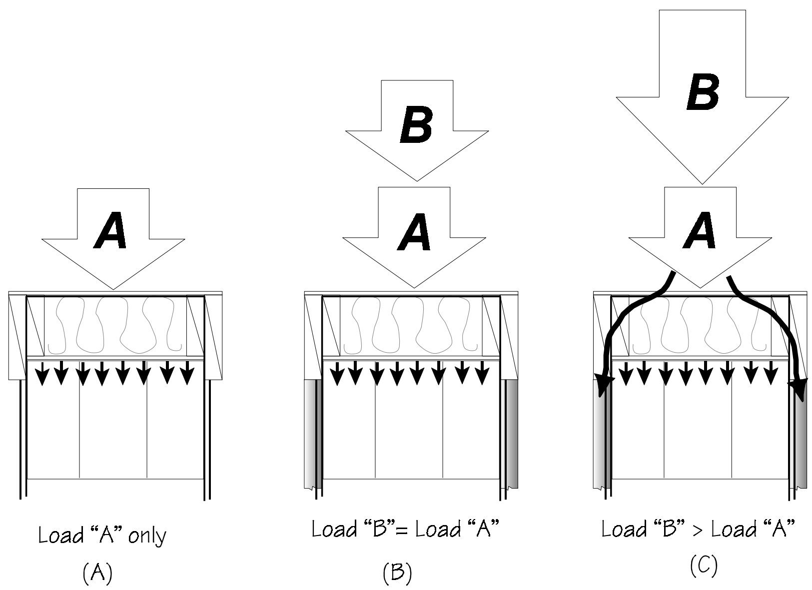

Fig.7: Load Transfer into Wall System

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 the design process that was used for the Seigel/Cochrane residence.

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 750 psi (5.17MPa) . A value of 300 psi (2.06 MPa) will be used for theoretical values.

ii) Stucco skins that are 1-1/2" thick on each side. Reinforced with 16 ga. galvanized 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 16 - inches on centre each way linking the two skins together with straw sandwiched in between.

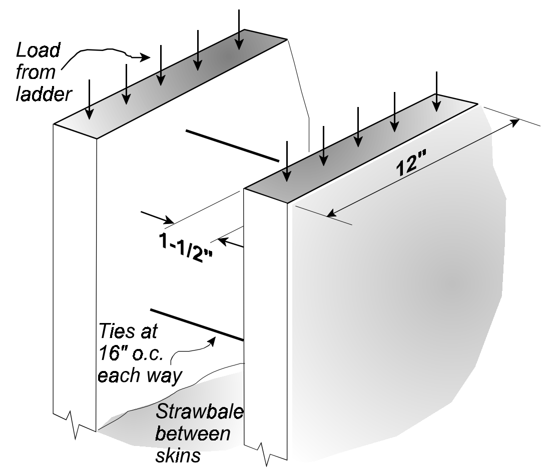

Fig. 8: Load Applied to Stucco Skin

Using the assumption of 300 psi for the compressive strength of the mortar in conjunction with a 1-1/2" stucco skin on each side of the bale wall a theoretical strength may be determined as follows for one lineal foot of wall.

For every lineal foot of wall there is:

$$\left(\frac{1.5in.}{12in.} \times 1ft \right) \times 2\ skins = 0.25 ft^2 \,/\, ft\ of\ \ wall \qquad (1)$$

Based on a 300 psi compressive strength,

$$300\frac{lbs}{in^2} \times 144\frac{in^2}{ft^2} = 43200\frac{lbs}{ft^2} \qquad (2)$$

A theoretical vertical load-carrying capacity may be determined as:

$$TheoreticalValue = 43200\frac{lbs}{ft^2} \times 0.25\frac{ft^2}{ft\ of\ wall} = 10{,}800\frac{lbs}{ft} \qquad (3.)$$

Figure 9 contains a plot of vertical load values obtained from various test results and theoretical calculations. The design value used for the Seigel/Cochrane residence was 2000 pounds per lineal foot(plf). Based on the data contained in Fig. 9, this represents approximately the 40th percentile. The largest factored wall load encountered in the design of the residence was 1200 plf. This represents approximately 60% of the design value. It should be noted that research data obtained after the design of the structure has been included in Fig. 9. Tests conducted at the University of Manitoba in January of 2002 indicate that the design value used for the Seigel/Cochrane residence is considered to be acceptable.

Lateral stability of the stucco skins is considered to be provided by the ties that run through the bale wall, connecting the wire mesh on each side of the wall together. As noted earlier, these ties are spaced at a maximum of 16-inches on centre each way(Fig.10). A nominal load of 300 plf is considered to be carried by the compressed bales. This leaves 900 plf, or 450 plf

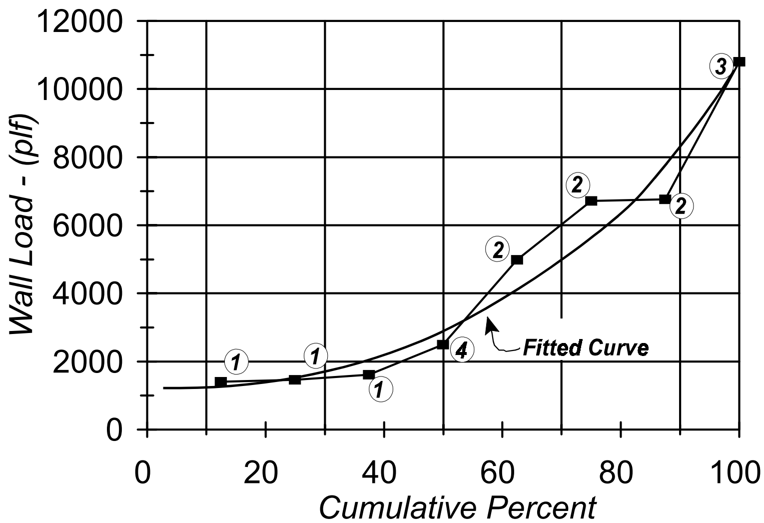

Fig.9: Vertical Design Load Plot
(Source:
Î - Grandsaert, 1999. Ï Carrick, 1998. Ð Theoretical Ñ Dick, 2002)

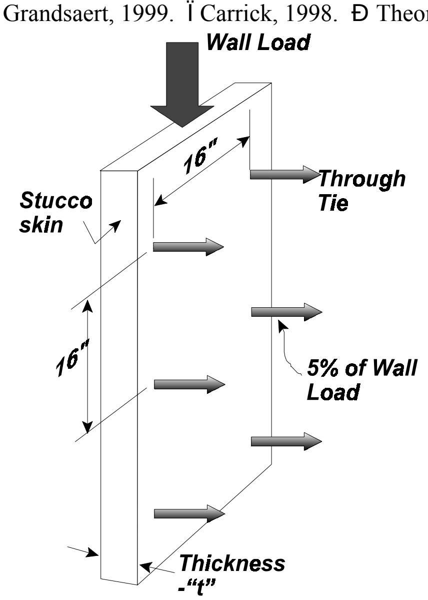

Fig.10. Lateral Resistance

along each stucco skin. Based on a 16-inch tie spacing and a 5% lateral force[1] the ties are required to provide 30 pounds 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 16-inch on centre, the slenderness ratio is then determined to be:

$$Slenderness\ Ratio = \frac{l}{0.3h} = \frac{16in.}{(0.3 \times 1.5in.)} = 35.5 \qquad (4.)$$

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). Based on Section 4.1.8 of the NBCC-1995 the reference velocity pressure for the Orillia, Ontario area is given as: $q_{1/30}$ = 0.32 kPa (6.68 psf).

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

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

[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.

[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.

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

$$p_i = q \times C_e \times C_g \times C_{pi}$$

$$p_i = 0.32 \times 0.9 \times 1.0 \times 0.7 = 0.2 kPa \, (4.2 psf) \qquad (6)$$

Based on the above loading, the wall is subjected to a total of 12 psf. Using a one-foot strip of the wall, a uniformly-distributed load of 12 pounds per lineal foot (plf) was used to determine the simply-supported bending moment in the wall. In the determination of the flexural stress the moment of inertia of the wall assembly must be considered. The moment of inertia used in the design of this structure was calculated to include the stucco skins and a portion of the straw as acting as a unit. It is apparent from both research and anecdotal evidence that there is a significant bond established between the stucco and the straw. For the purposes of a design check, however, it was considered appropriate to use only the stucco skins in the moment of inertia calculation. The moment of inertia for the section was determined to be:

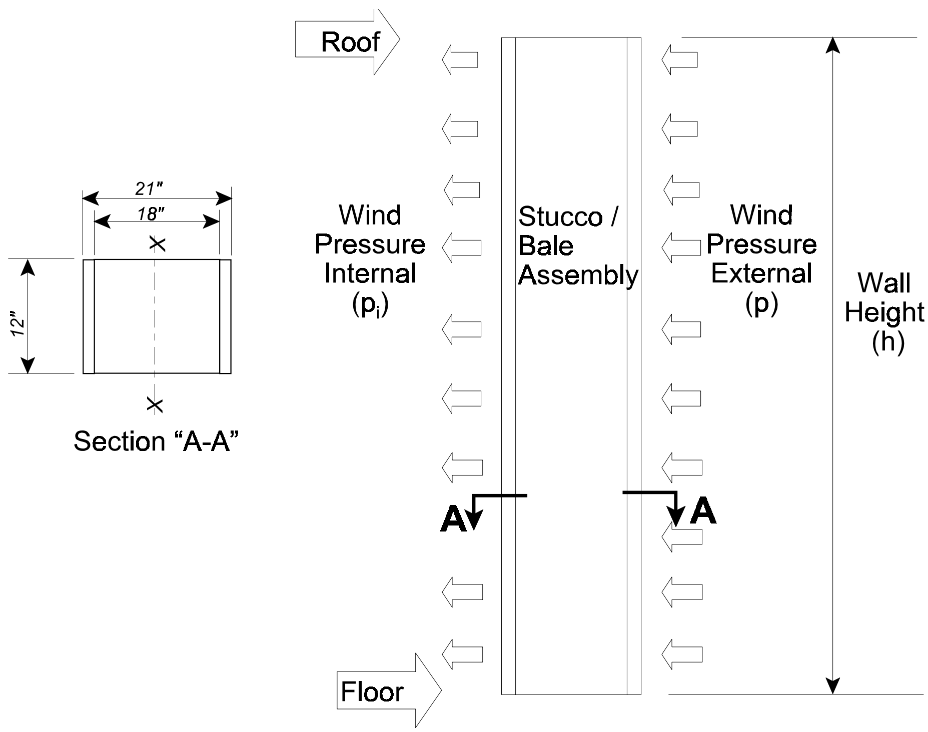

Fig.11: Lateral Wind Load on Wall System

$$I = \left(\frac{bd_1^3}{12}\right) - \left((\frac{bd_2^3}{12})\right) = \left(\frac{12 \times 21^3}{12}\right) - \left((\frac{12 \times 18^3}{12})\right) = 3429 \, in^4 \quad (7)$$

[3] Based on a Category 2 structure

Based on the following assumptions:

- a wall height of eight (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:

$$\sigma_f = \frac{My}{I} = \frac{\left((\frac{1.5 \times 12 \times (8)^2}{8}) \times 12 \right) \times \frac{21}{2}}{3429} = 5.3 psi \qquad (8)$$

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 allowable stress in tension would be 300 psi x 0.05 = 15 psi. Thus the design meets this criterion. If the straw component is included in the moment of inertia calculation the flexural stress becomes 2.9 psi.

The mid-height deflection was calculated based on an moment of inertia that only included the stucco skins. A composite modulus of elasticity E_{comp} for the stucco / straw assembly was taken to be 82,650 psi (570 MPa) based on relative areas of straw and stucco. The mid-height deflection was calculated based on a simply-supported flexural member and found to be:

$$\Delta = \frac{5wl^4}{384EI} = \frac{5 \times 12 \times (8 \times 12)^4}{384 \times 82{,}650 \times 3429} = 0.047\ in. \qquad (9)$$

This value corresponds to a deflection with respect to wall height of L/2042, well within acceptable limits.

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
researched based on both shear flow (force per unit length - plf) and shear stress (force per unit area - psf).

The following represent examples of typical values for these shear parameters:

Shear Flow: - 750 plf (White and Iwanicha, 1997)
- 1005 plf (Boynton, 1999)

Shear Stress: - 42 psf (Riley et al., 1998)
- 60 psf (Dick and Britton, 2002) [4]

Within the context of the Seigel / Cochrane project the factored shear at the stucco - strawbale bond for the wind load discussed in the previous section was determined to be 27.21 plf (shear flow) or 0.1 psi (shear stress) for a 1-foot wide strip of wall.

$$Shear\ Flow = f = \frac{VQ}{I} = \left(\frac{48lbs \times 162in^3}{3429in^4}\right) \times 12\frac{in}{ft} = 27.21\,plf \qquad (9)$$

$$Shear\ Stress - \tau = \frac{VQ}{Ib} = \left(\frac{48lbs \times 162in^3}{3429in^4 \times 12in}\right) = 0.19\,psi \qquad (10)$$

These values are considerably below test results and were considered to be acceptable for the design of the Seigel / Cochrane structure.

4. Conclusion:

The basis for key aspects for the design of the Seigel / Cochrane strawable residence have been presented in this report. Based on theoretical calculations and research data it is the opinion of the designers that the structural integrity of the strawable building discussed herein will meet or exceed the requirements for imposed loading for the Seigel / Cochrane residence in Severn Township, Orillia, Ontario.

Report prepared and submitted by:
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KD/kjd

cc: -file
- J Seigel and C. Cochrane

[4] Results of more recent testing have been included in this report although original designs were done before these data were available. Recent research results have been presented in this report to provide additional background.

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