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688-Straw Bale Code

SUPPLEMENTAL

STRUCTURAL CALCULATIONS

IN SUPPORT OF

ICC CODE CHANGE
CHAPTER 24
STRAWBALE CONSTRUCTION

OUT-OF-PLANE LOADS
(PROPOSED CODE SECTIONS
2405.11 & 2405.12)

FEBRUARY 29, 2012

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SUPPLEMENTAL CALCS
ICC CODE CHANGE
STRAW BALE CONSTRUCTION

JOB NO. ______ SH ______
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BY KBD DATE 2/29/12

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OUT-OF-PLANE TEST OF 8' x 8' WALL PANEL AND ANALYSIS

JUNE 25, 2001
REVISED SEPTEMBER 17, 2002
7 PAGES

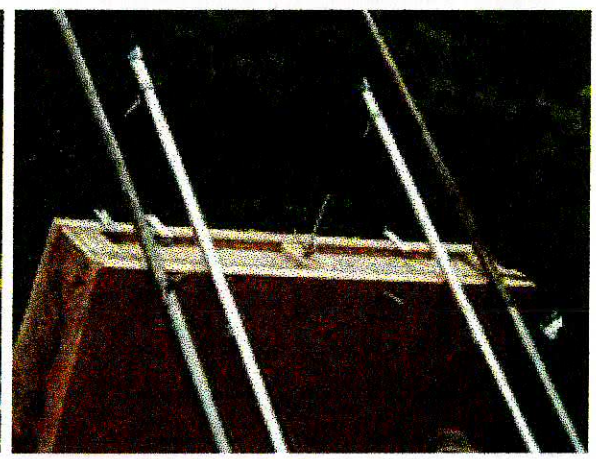
Preliminary Report on the Out-of-plane Testing of an 8'x8' Straw Bale/PISE Wall Panel

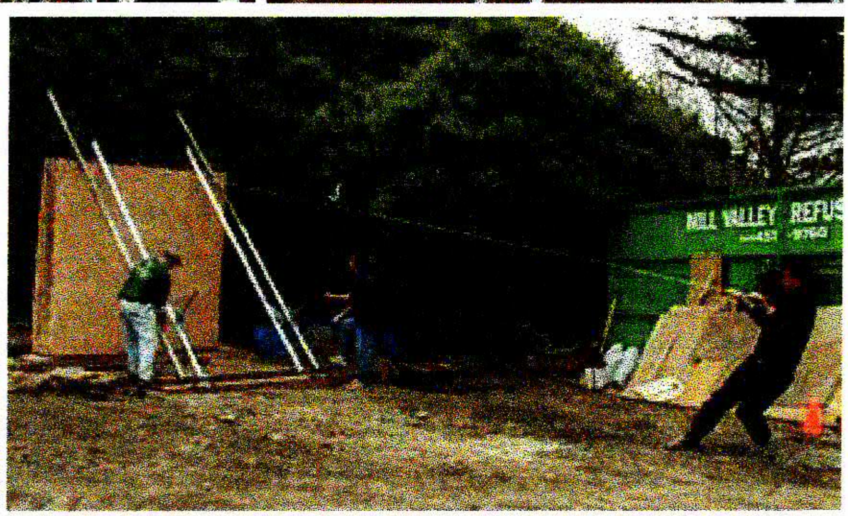
KEVIN DONAHUE AND DAVID ARKIN - 6/25/01

The tests were conducted at the Breeze Residence in Mill Valley, CA in conjunction with the construction of the Breeze Residence, whose walls included straw bale/PISE walls of similar construction to the test panel.

The tests were conducted by David Arkin of Arkin-Tilt Architects of Albany, CA—the project architect—and Kevin Donahue of Toft, de Nevers & Lee of San Francisco, CA—the project structural engineer, with help from Rob Nelson, the contractor Neal McDonald, Bruce King and numerous other straw bale enthusiasts.

The tests were conducted on March 24, 2001, when the panel was lowered from its initial vertical position to a horizontal position using a system of hinges and wall jacks, and April 11, 2001, when the horizontal panel was loaded with water placed in a 6.75'x6.75'x3' visqueen covered wood frame centered on top of the panel.

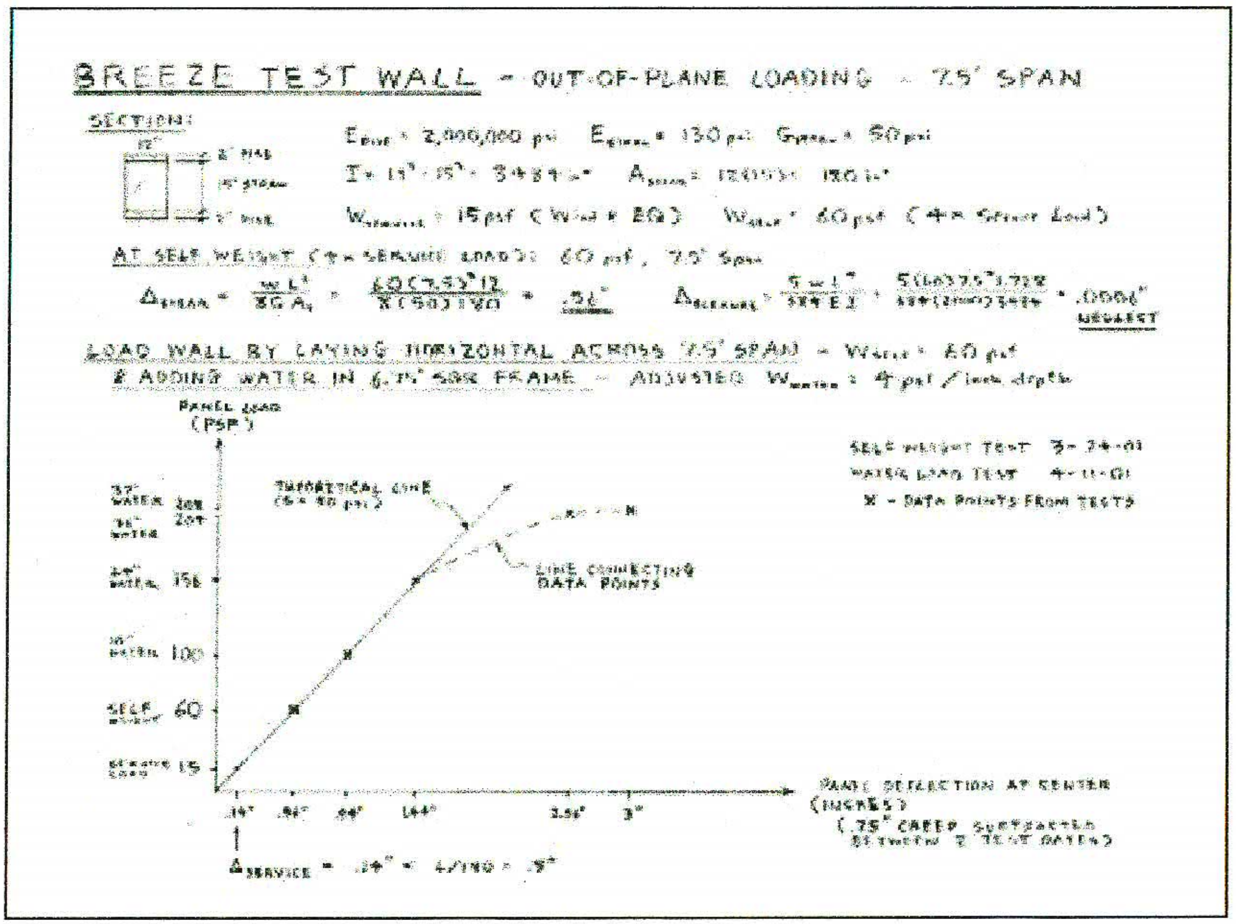




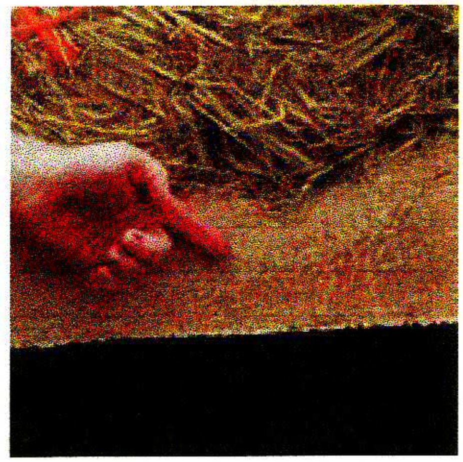
The wall panel's construction consisted of 15" three-string rice straw bales laid on edge in a running bond with wood I-joists top and bottom and 2" PISE veneer both sides with a 2"x3"x16GA wire mesh (3" spacing vertical) attached top and bottom with 16GA staples @ 3". The bales were unpinned except for 1/2" rods at 2'-0" to the base course, and contained no through ties, exterior pinning or any other supports in addition to those outlined above. The PISE (pneumatically impacted stabilized earth) was "Nunn's Canyon 7-2-1 Mix"—7 parts Nunn's Canyon Soil, 2 parts sand and 1 part portland cement—with a specified compression strength of 1000psi and an average cylinder-tested strength of 2620psi (cylinder testing by Inspection Services of San Francisco, CA).

On March 24 the panel was placed in its horizontal position spanning 7.5' between supports, and weighed with a dynamometer supplied by Testing Engineers of Oakland, CA. The measured gravity self-weight of 60psf represents a load of four times the service load of 15psf from wind or earthquake loads that the panel would receive in its vertical position. (It should be noted that there is a conservatism in this test because the beneficial effects of gravity on the frictional resistance to out-of-plane deformation is lost when the panel is moved from the vertical to the horizontal position.) Deflection at the center of the wall was measured by noting the movement of an embedded ruler relative to a wire pulled taut from supports at the panels ends.

On April 11, a superimposed load of water in a 6.75'x6.75' wood frame was applied to the panel, and corresponding deflections were measured. The results of both phases of the test are indicated on the accompanying graph with selected data points shown. It should be noted that .25" of creep (time dependant movement) occurred between the two test dates, and this .25" has been subtracted from the water load results from April 11.



Conclusions:



The panel remained stiff and well behaved for loads greater than ten times the 15psf service load. A structural element is generally considered safe if its ultimate load is over four times the anticipated service load, and by this measure the straw bale/PISE panel performed extremely well. Even better performance would be expected from similarly constructed thicker walls.

The panel behaved as a true sandwich panel with fully composite action between the PISE flanges and the straw bale web, with virtually all deflection being due to shear deformation of the straw bale web. However, it should be noted that the perfect correspondence between the calculated and measured performance of the panel depends on the assumption that the straw bale's shear modulus G is 50psi. Clearly, this assumption is an item of further study, and perhaps careful measurement in a future test.

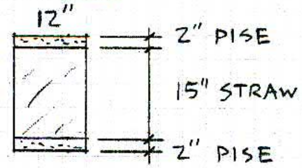
Finally, deflection of the panel under the 15psf can be interpolated to be 0.14", which is considerably less than the maximum allowable wall deflection of L/180=0.5". By this stiffness standard the panel's performance can also be deemed successful.

OUT-OF-PLANE TEST OF 15 × 7.5 STRAW BALE WALL W/ HARD SKIN

REVISED 9-17-02

BREEZE TEST WALL - OUT-OF-PLANE LOADING - 7.5' SPAN

SECTION:



$E_{PISE} = 2{,}000{,}000$ psi $E_{STRAW} = 130$ psi $G_{STRAW} = 50$ psi

$I = 19^3 - 15^3 = 3484$ in^4 $A_{SHEAR} = 12(15) = 180$ in^2

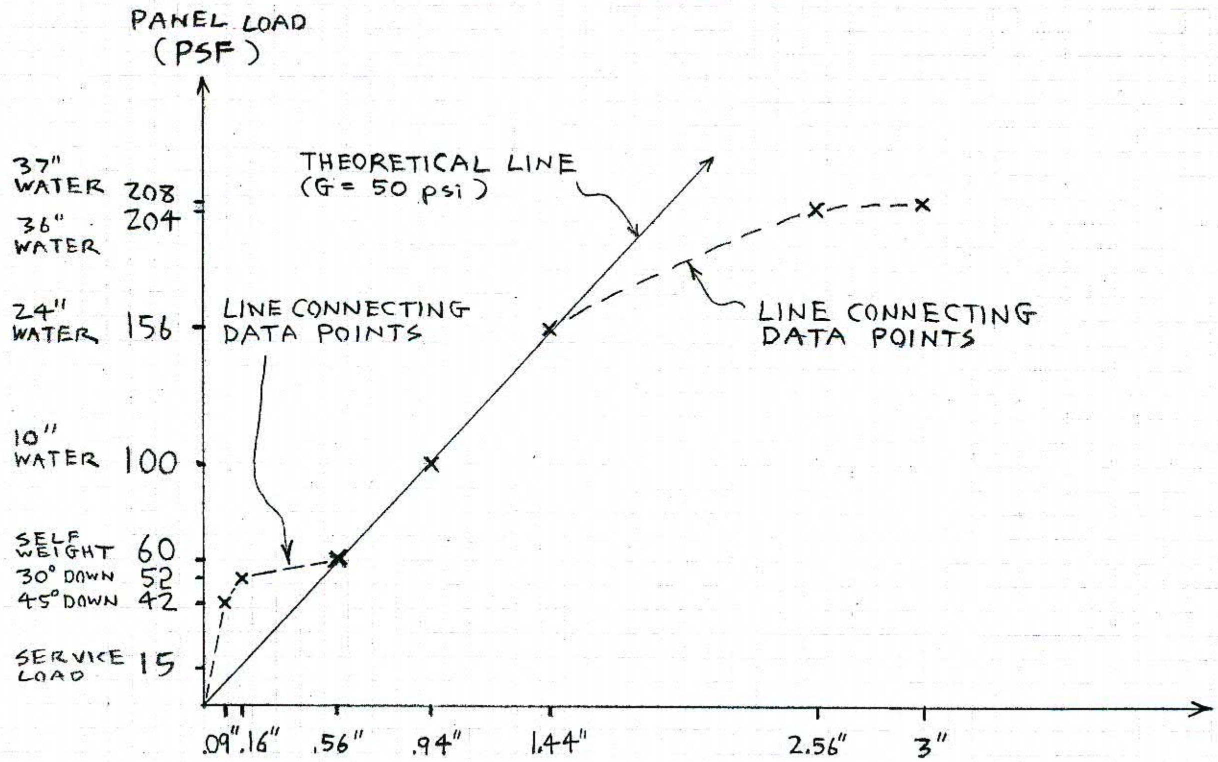
$W_{SERVICE} = 15$ psf (Wind & EQ) $W_{SELF} = 60$ psf (4 × Service Load)

AT SELF WEIGHT (4 × SERVICE LOAD): 60 psf, 7.5' Span

$$\Delta_{SHEAR} = \frac{wL^2}{8GA_s} = \frac{60(7.5)^2 12}{8(50)180} = \underline{.56''} \qquad \Delta_{FLEXURE} = \frac{5wL^4}{384EI} = \frac{5(60)7.5^4 1.728}{384(2000)3484} = .0006''$$

NEGLECT

LOAD WALL BY LAYING HORIZONTAL ACROSS 7.5' SPAN - $W_{SELF} = 60$ psf & ADDING WATER IN 6.75' SQR FRAME - ADJUSTED $W_{WATER} = 4$ psf / inch depth



PANEL DEFLECTION AT CENTER (INCHES)
(.25" CREEP SUBTRACTED BETWEEN 2 TEST DATES)

SELF WEIGHT TEST 3-24-01
WATER LOAD TEST 4-11-01
X - DATA POINTS FROM TESTS

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$$\Delta_{SERVICE} = \frac{15}{42}.09'' = .03'' < L/180 = .5''$$

$$\Delta_{SERVICE\ (THEORETICAL\ LINE)} = \frac{15}{60}.56'' = .14'' < L/180 = .5''$$

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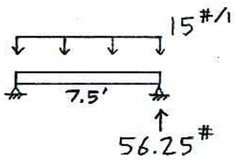
BREEZE TEST WALL

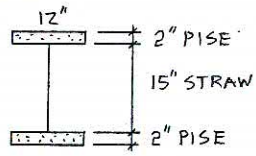
JOB NO. ______ SH. ______
NO. 2 OF 3
BY KED DATE 9-15-02

LOAD TEST WALL WITH SERVICE LOAD $W_{SERVICE} = 15$ psf

$E_{PISE} = 2{,}000{,}000$ psi $E_{STRAW} = 130$ psi $G_{STRAW} = 50$ psi
$(E/2(1+.3))$ $\nu = .3$



SECTION:



TRANSFORMED WIDTH OF STRAW:

$b = 12''(130/2{,}000{,}000) = .00078''$ (NEGLECT)

$I = 19^3 - 15^3 = 3484 \text{ in}^4$

$S = 3484/9.5 = 366.7 \text{ in}^3$

STIFFNESS OF PRIMARY SYSTEM LOAD w/ W_p, $\Delta_{FLEX} = \frac{5}{384}\frac{W_p L^4}{EI}$, $\Delta_{SHEAR} = \frac{W_p L^2}{8 G A_{WEB}}$

$\Delta = \frac{5}{384}\frac{W_p\, 7.5^4 (12)^4}{2{,}000{,}000(3484)} + \frac{W_p\, 7.5^2 (12)^2}{8(50)180} = .000123\, W_p + .1125\, W_p = .1126\, W_p$

$W_p = K_p \Delta$ $K_p = 8.88^{\#/\text{in}^2}$

STIFFNESS OF SECONDARY SYSTEM - 2 SKINS LOAD w/ W_s, $\Delta = \frac{5}{384}\frac{W_s L^4}{E I_{SKINS}}$

$I_{SKINS} = 2(2^3) = 16 \text{ in}^4$ $\Delta = \frac{5}{384}\frac{W_s\, 7.5^4 (12)^4}{2{,}000{,}000(16)} = .0267\, W_s$

$W_s = K_s \Delta$ $K_s = 37.46^{\#/\text{in}^2}$

$W_T = W_p + W_s = (K_p + K_s)\Delta = K_T \Delta$ $K_T = 8.88 + 37.46 = 46.34^{\#/\text{in}^2}$

LOAD PRIMARY SYSTEM $W_p = (8.88/46.34)15^{\#/1} = 2.87^{\#/1}$

$\Delta = .1126(2.87/12) = .027''$

$M = 2.87^{\#/1}(7.5')^2/8 = 20.2^{\#'}$ $f_b = 20.2(12)^{\#''}/366.7 \text{ in}^3 = .66$ psi

$V = 2.87(3.75) = 10.76^{\#}$ AT STRAW-PISE INTERFACE

$f_v = \frac{VQ}{Ib} = 10.76^{\#}(8.5'')24^{\text{in}^2}/3484 \text{ in}^4 (12'')$

$= .053$ psi $(7.56$ psf$) < 3$ psi $(432$ psf$)$

LOAD SECONDARY SYSTEM $W_s = (37.46/46.34)15^{\#/1} = 12.13^{\#/1}$

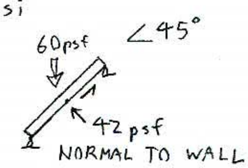
$\Delta = .0267(12.13/12) = .027''$ (Agrees w/ Above)

$M = 12.13^{\#/1}(7.5')^2/8 = 85.3^{\#'}$ $f_b = 85.3(12)^{\#''}(1'')/16 \text{ in}^4 = 63.97$ psi

$V_{PISE} = 12.13(3.75) = 45.5^{\#}$ $f_v = 1.5(45.5)/12''(4)'' = 1.4$ psi

COMBINE SYSTEMS PISE $f'_c = 2500$ psi $f_{crack} = 4\sqrt{f'_c} = 200$ psi

$f_t = .66 + 63.97 = 64.6$ psi < 200 psi, OK



CHECK WALL AT 45° $W = W_{SELF}/\sqrt{2} = 60^{\#/1}/\sqrt{2} = 42^{\#/1}$

FROM ABOVE $\Delta = (\frac{42}{15}).027'' = .076''$ (Measured $= .09''$)

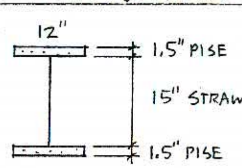
$f_{v\,STRAW} = (\frac{42}{15}).053 = .148$ psi $(21.3$ psf$)$

$f_{v\,PISE} = (\frac{42}{15})1.4 = 3.9$ psi

$f_t = (\frac{42}{15})64.6 = 181$ psi ≈ 200 psi - Incipient Cracking - Agrees w/ Test (See Graph)

CHECK FOR CRACK PROPAGATION - PROGRESSIVE LOSS OF SKIN STIFFNESS

TEST SECTION:



FOLLOWING SIMILAR PROCEDURE AS ABOVE YIELDS:

$f_t = 257$ psi > 181 psi

CRACK PROPAGATES - SKIN STIFFNESS LOST

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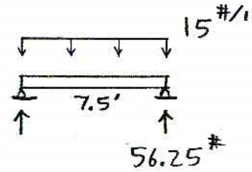
BREEZE TEST WALL

NOTE: SUPERSEDED BY PREVIOUS PAGE – THESE BENDING VALUES REFER TO THE UNIFORM STRESS ON THE SKINS AND DON'T INCLUDE SECONDARY BENDING – THESE VALUES WOULD APPLY TO ULTIMATE STRESS ON A TOUGH PLASTER/FIBER MATRIX ABLE TO ABSORB THE SECONDARY STRESS

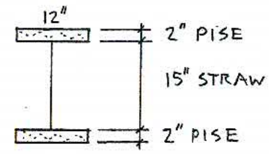
LOAD TEST WALL WITH SERVICE LOAD

$W_{SERVICE} = 15$ psf

$E_{PISE} = 2{,}000{,}000$ psi $E_{STRAW} = 130$ psi $G_{STRAW} = 50$ psi



SECTION:



TRANSFORMED WIDTH OF STRAW:

$b = 12''(130/2{,}000{,}000) = .00078''$ (NEGLECT)

$I = 19^3 - 15^3 = 3484$ in⁴

$S = 3484/9.5 = 366.7$ in³

$M = 15^{\#/'}(7.5')^2/8 = 105.5^{\#'}$ $f_b = 105.5(12)^{\#''}/366.7\text{ in}^3 = 3.45$ psi

$V = 56.25^{\#}$ AT STRAW-PISE INTERFACE $f_v = \frac{VQ}{Ib} = 56.25^{\#}(8.5'')24^{in^2}/3484^{in^4}(12)''$

$= .274$ psi (39.5 psf)

$\Delta_{FLEX} = \frac{5}{384}\frac{WL^4}{EI} = \frac{5(15)\,7.5^4\,1.728}{384(2000)\,3484} = .000015''$ (NEGLECT)

$\Delta_{SHEAR} = \frac{WL^2}{8GA_{WEB}} = 15(7.5)^2 12/8(50)180 = .1406''$

LOAD WALL WITH 10 × SERVICE LOAD

$W_{10x} = 150$ psf

ASSUME UNCRACKED SECTION Assume PISE $f'_c = 2500$ psi $f_{CRACK} = 4\sqrt{f'_c} = 200$ psi

$f_b = 10(3.45) = 34.5$ psi $f_v = 10(.274) = 2.74$ psi (395 psf)

ASSUME CRACKED SECTION

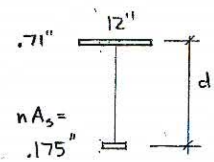
STEEL REINF. IN PISE – 16GA @ 3" $f_y = 60$ ksi $A_s = 4(.00302) = .0121$ in²

$nA_s = (29{,}000/2{,}000).0121 = .175$ in²

CHECK CENTROID

$y_c A_c = .355(.71)12 = 3.025$ in³

$y_t A_t = 17.29(.175) = 3.025$ in³



$I = 12(.71)^3/3 + .175(17.29)^2 = 53.75$ in⁴

$S_{PISE} = 53.75/.71 = 75.7$ in³

$S_{STEEL} = (53.75/17.29)(2{,}000/29{,}000) = .214$ in³

$M = 150(7.5)^2/8 = 1055^{\#'}$ $f_{PISE} = 1055(12)/75.7 = 167$ psi Compression < 2500

$f_{STEEL} = 1055(12)/.214 = 59{,}160$ psi Tension < 60,000

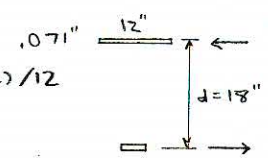
$V = 562.5^{\#}$ $f_v = 562.5(.355)(.71)12/53.75(12) = 2.64$ psi (380 psf) < 3 psi (432 psf) Straw

(NOTE: ASSUME WALL IS CRACKED ALONG ENTIRE LENGTH – VERY CONSERVATIVE)

$\Delta_{FLEX} = \frac{5(150)\,7.5^4\,1.728}{384(2000)\,53.75} = .099''$ (STILL NEGLECT !)

$\Delta_{SHEAR} = 150(7.5)^2 12/8(50)180 = 1.406''$

ULTIMATE STRENGTH OF CRACKED SECTION



$C = .85(1000)12(.071) = 726^{\#}$

$T = 60{,}000(.0121) = 726^{\#}$

$M_n = 762(18 - .071/2)/12 = 1141^{\#'}$

$W_u = 1141(8)/7.5^2 = 162^{\#/'}$ (10.8 × Service Load)

(See Graph – Additional Strength due to Steel strain hardening)

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OUT-OF-PLANE LOADS
(PROPOSED CODE SECTIONS 2405.11 & 2405.12)

BEAM THEORY APPLED TO OUT-OF-PLANE LOADING

SEPTEMBER 17, 2002
4 PAGES

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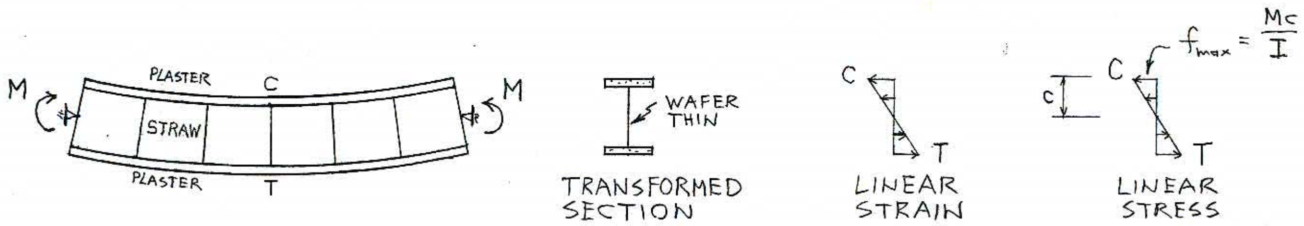
APPLY CLASSIC BEAM THEORY TO OUT-OF-PLANE STRAW BALE WALL

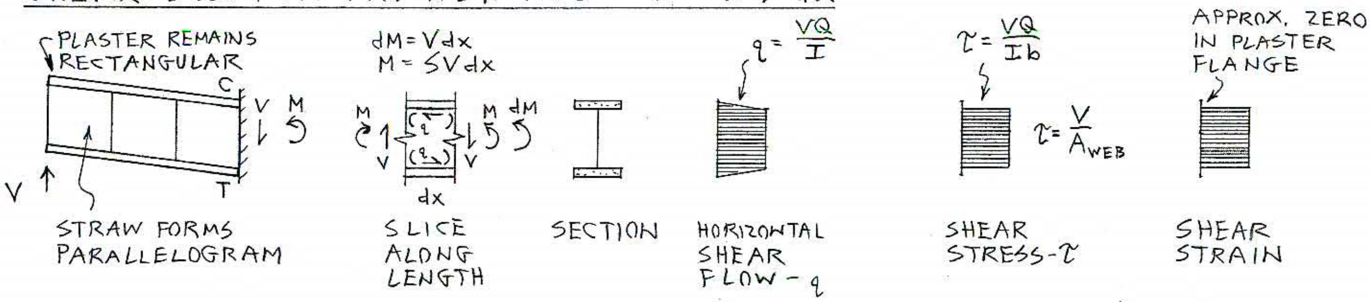
NOTE: ASSUME WALL BEHAVES AS CLASSIC BEAM UNLESS PROVEN OTHERWISE

SUPERPOSE FLEXURAL BEHAVIOR AND SHEAR BEHAVIOR

FLEXURE SOLUTION FROM KINEMATIC ASSUMPTION: PLANE SECTIONS REMAIN PLANE



SHEAR SOLUTION FROM STATICS: $V = dM/dx$



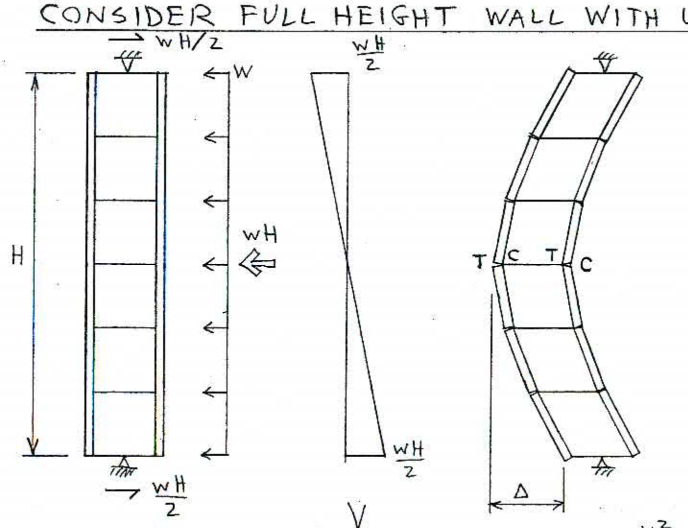
NOTE: FLEXURAL STRESS DERIVED FROM STRAIN
SHEAR STRAIN DERIVED FROM STRESS

SUBSEQUENT SHEAR DISTORTION IS COMPLEX, BUT GENERALLY A SECOND-ORDER EFFECT, SINCE SHEAR DEFORMATION IS TYPICALLY MUCH LESS THAN FLEXURAL

IN THE CASE OF STRAW BALE OUT-OF-PLANE, THE REVERSE IS TRUE, WITH DISPLACEMENT FROM SHEAR ABOUT ONE-THOUSAND TIMES LARGER THAN DISPLACEMENT FROM FLEXURE, DUE TO THE EXTREME DISPARITY BETWEEN THE STIFFNESS OF THE PLASTER FLANGES AND THE STRAW WEB

THE FACT THAT THE OVERALL SHEAR SECTION DOES NOT FORM A PARALLELOGRAM DUE TO SHEAR DEFORMATION CAUSES AN INCOMPATABILITY IN THE OUT-OF-PLANE WALL WHEN ALL THE PIECES ARE ASSEMBLED

CONSIDER FULL HEIGHT WALL WITH UNIFORM OUT-OF-PLANE LOAD



TO OBEY CLASSIC BEAM THEORY (WITH SHEAR STRAIN DOMINATING FLEXURAL STRAIN BY A FACTOR OF 1,000), THE STRAW TENDS TO FORM PARALLELOGRAMS AND THE PLASTER SKINS TEND TO FORM RECTANGLES. WHEN THE PIECES ARE ASSEMBLED TO MAKE UP THE WHOLE WALL, THERE IS AN INCOMPATABILITY WITH THE STIFF PLASTER SKIN RESISTING THIS TENDENCY. THIS SECONDARY STIFFNESS OF THE PLASTER SKINS CAN BE APPROXIMATED BY MODELING THE 2 SKINS AS SECONDARY BEAMS BENDING ON THEIR INDIVIDUAL WEAK AXES.

$\Delta = \frac{wH^2}{8GA_{WEB}}$ (ASSUMING SKIN IS CRACKED AS SHOWN, BUT STILL CABABLE OF TAKING OVERALL C-T FLEXURAL COUPLE FOR WHOLE WALL)

EGOR P. POPOV INTRODUCTION TO MECHANICS OF SOLIDS

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eam deflection due

limited to those in
curved beams is
hapter only beams
the applied forces
symmetry and the
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be considered.

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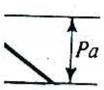

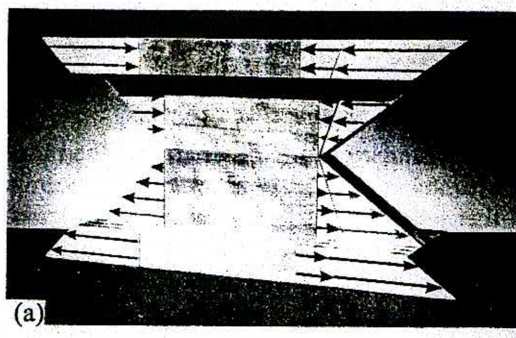

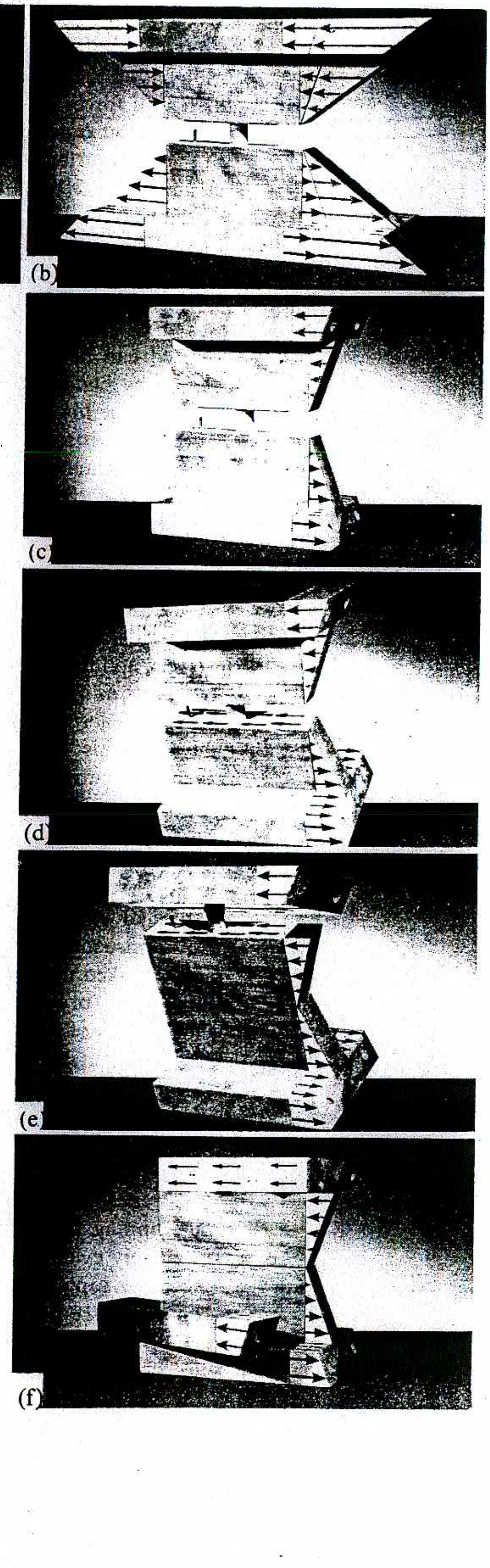

change in the bending moment occurs. Conversely, the rate of change of the bending moment along the beam is numerically equal to the shear. Thus, although shear is treated in this chapter as an independent action on a beam, it is inseparably linked with a change in the bending moment along the length of the beam.

To emphasize the meaning of Eq. 7-1 further, the shear and bending diagrams of Example 2-9 are reproduced in Fig. 7-1. At any two sections such as A and B taken through the beam anywhere between the applied forces P, the bending moment is the same. No shear acts at these sections. On the other hand, at any two sections such as C and D near the support, a change in the bending moment does take place. Shearing forces act at these sections as in Fig. 7-1(d). In the subsequent discussion, the possibility of equal, as well as of different, bending moments on two adjoining sections through a beam must be appreciated.

Next, before the detailed analyses, a study of a sequence of photographs of a model (Fig. 7-2) may prove helpful. The model represents a segment of an I beam. In Fig. 7-2(a), in addition to the beam itself, blocks simulating stress distribution caused by bending moments may be seen. The

Fig. 7-2. Shear flow model of an I beam. (a) Beam segment with bending stresses shown by blocks. (b) Shearing force transmitted through dowel. (c) For determining the force on a dowel only change in moment is needed. (d) The shearing force divided by the area of the cut yields shearing stress. (e) Horizontal cut below the flange for determining the shearing stress. (f) Vertical cut through the flange for determining the shearing stress.

APPENDIX B-2
3 OF 4

SOLUTION

From the point of view of elasticity, internal stresses and strains in beams are statically indeterminate. However, in the technical theory discussed here, the introduction of a kinematic hypothesis that plane sections remain plane after bending changes this situation. Here, in Eq. 6-3, it is asserted that in a beam $\sigma_x = -My/I$. Therefore, one part of Eq. 3-3—that giving the differential equation of equilibrium for a two-dimensional problem with a body force $X = 0$—suffices to solve for the unknown shearing stress. From the conditions of no shearing stress at the top and the bottom boundaries, $\tau_{yx} = 0$ at $y = \pm h/2$, the constant of integration is found.

From Eq. 3-3: $$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} = 0$$

But $$\sigma_x = -\frac{My}{I}, \quad \text{hence} \quad \frac{\partial \sigma_x}{\partial x} = -\frac{\partial M}{\partial x}\frac{y}{I} = \frac{Vy}{I}$$

and Eq. 3-3 becomes $$\frac{Vy}{I} + \frac{d\tau_{xy}}{dy} = 0$$

Upon integrating $$\tau_{xy} = -\frac{Vy^2}{2I} + C_1$$

Since $$\tau_{xy}(\pm h/2) = 0, \quad \text{one has} \quad C_1 = +\frac{Vh^2}{8I}$$

and $$\tau_{xy} = \tau_{yx} = +\frac{V}{2I}\left[\left(\frac{h}{2}\right)^2 - y^2\right]$$

This agrees with the result found earlier since here $y = y_1$.

According to Hooke's law, shearing deformations must be associated with shearing stresses. Therefore the shearing stresses given by the above relation must cause shearing deformations. As shown in Fig. 7-12, maximum shearing distortions occur at $y = 0$, and no distortions take place at $y = \pm h/2$. This warps the initially plane section through the beam and contradicts the basic assumption of the technical bending theory. However, by the methods of elasticity it can be shown that these shearing distortions of the plane sections are negligibly small for slender members; the technical theory is completely adequate if the length of a member is at least two to three times greater than its total depth. This conclusion is of far-reaching importance since it means that the existence of a shear at a section does not invalidate the expressions for bending stresses derived earlier. As noted before, at the point of load application as well as at a rigidly built-in end, additional local disturbances of stresses occur.

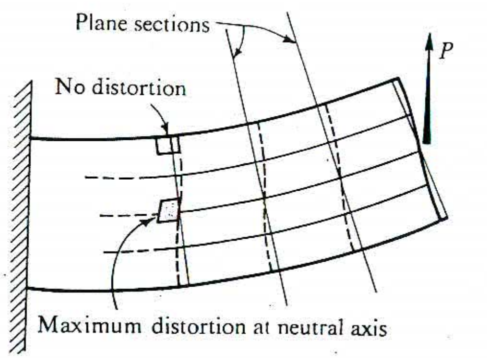

Fig. 7-12. Shearing distortions of a beam.

nal forces occurs

ıy using a more
the simultaneous
nental aspects of
ıapter 9.

ss section shown
levels indicated.

ig. 7-14(b). It is
ection is 50 kips.
nt problem. The
ıted in the table
ring stresses are
dths of the beam.

97 lb/in.[4]

t	τ, psi
6.0	0
6.0	−570
0.5	−6,800
0.5	−7,300
0.5	−9,780

quare inches.
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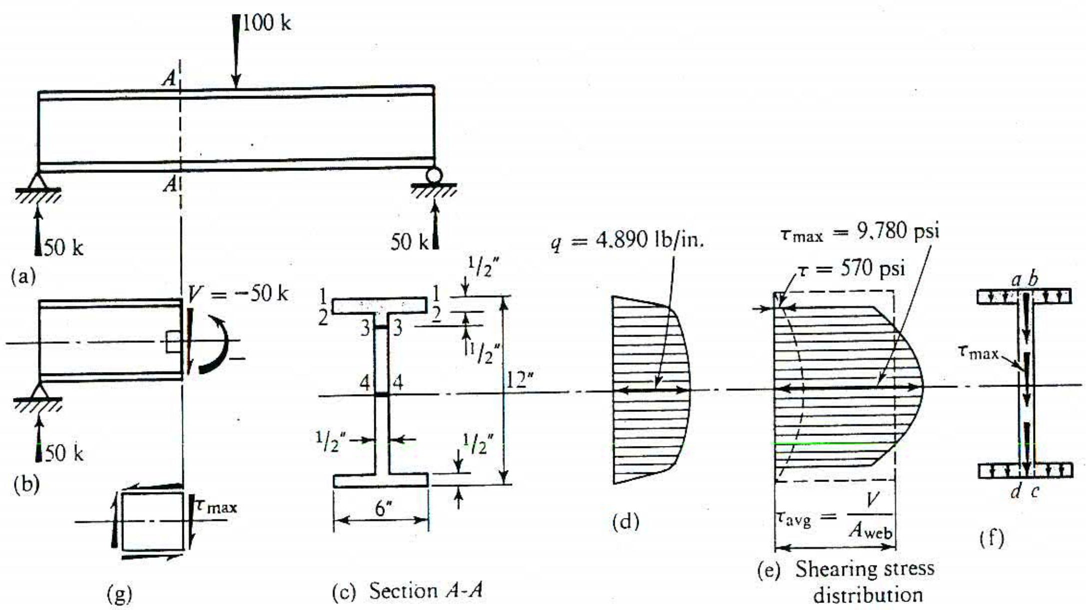

Fig. 7-14

Note that at the level 2-2 two widths are used to determine the shearing stress—one just above the line 2-2, and one just below. A width of 6 in. corresponds to the first case, and 0.5 in. to the second. This transition point will be discussed in the next article. The results obtained, which by virtue of symmetry are also applicable to the lower half of the section, are plotted in Fig. 7-14(d) and (e). By a method similar to the one used in the preceding example, it may be shown that the curves in Fig. 7-14(e) are parts of a second-degree parabola.

The variation of the shearing stress indicated by Fig. 7-14(e) may be interpreted as is shown in Fig. 7-14(f). The maximum shearing stress occurs at the neutral axis; the vertical shearing stresses throughout the web of the beam are nearly of the same magnitude. The shearing stresses occurring in the flanges are very small. For this reason the maximum shearing stress in an I beam is often approximated by dividing the total shear V by the cross-sectional area of the web (area $abcd$ in Fig. 7-14(f)). Hence

$$(\tau_{max})_{approx} = V/A_{web} \tag{7-9}$$

In the example considered this gives

$$(\tau_{max})_{approx} = \frac{50{,}000}{(0.5)12} = 8{,}330 \text{ psi}$$

This stress differs by about 15 per cent from the one found by the accurate formula. For most cross sections a much closer approximation

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688-Straw Bale Code

SUPPLEMENTAL STRUCTURAL CALCULATIONS

IN SUPPORT OF

ICC CODE CHANGE
CHAPTER 24
STRAWBALE CONSTRUCTION

OUT-OF-PLANE LOADS
(PROPOSED CODE SECTIONS 2405.11 & 2405.12)

OUT-OF-PLANE MODEL OF BALE WALL WITH HIGH INITIAL STIFFNESS

APRIL 16, 2006
3 PAGES

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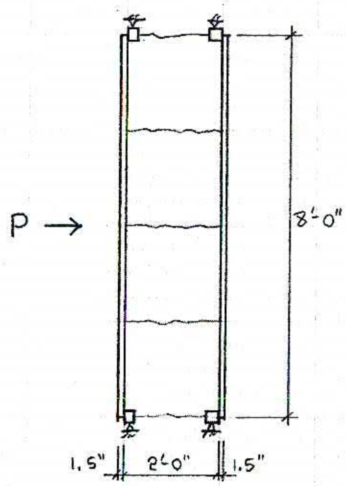
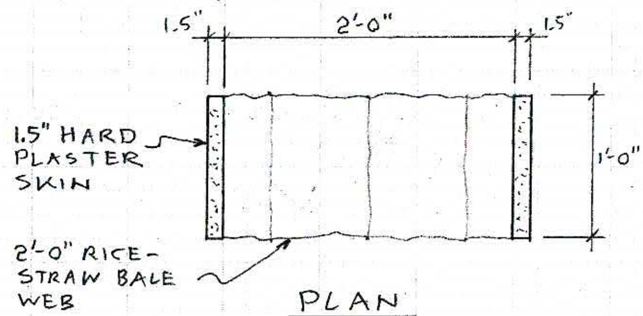
MODEL STRAW-BALE WALL	JOB NO. S/B	SH	
HARD SKIN UNCRACKED	NO. 1	OF 3	
HIGH INITIAL STIFFNESS	BY KED	DATE 4/16/06	

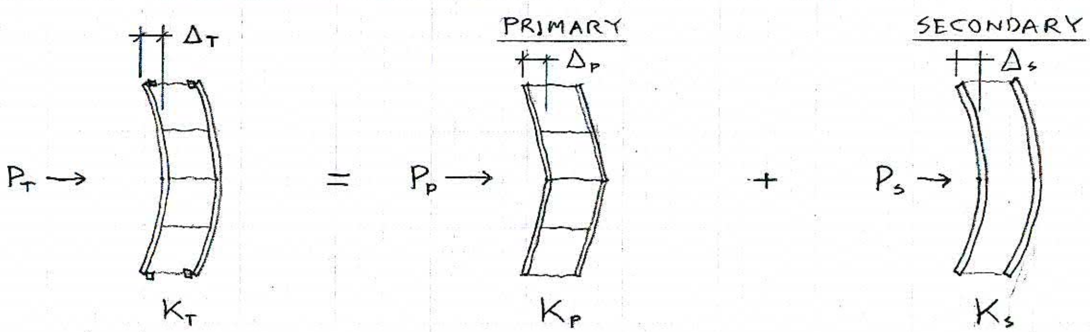
INITIAL BEHAVIOR OF MODEL STRAW-BALE WALL – HARD SKIN UNCRACKED





PLAN

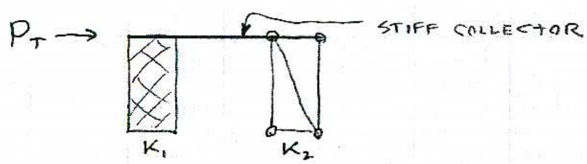
SUPERPOSE 2ND-ORDER INDIVIDUAL BENDING OF OUTER SKINS W/ GLOBAL DISPLACEMENT OF SECTION

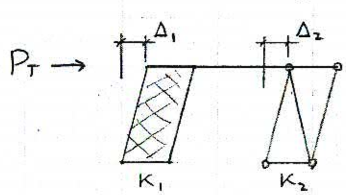


$P_T = K_T \Delta_T$ $\quad P_P = K_P \Delta_P$ $\quad P_S = K_S \Delta_S$ $\quad \Delta_T = \Delta_P = \Delta_S$

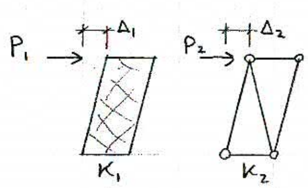
$\therefore K_T = K_P + K_S$

ANALOGOUS TO 2 IN-LINE SHEAR WALLS W/ STIFF COLLECTOR





$\Delta = \Delta_1 = \Delta_2$



APPORTION LOAD ACCORDING TO STIFFNESS

$K_T = K_1 + K_2$

GLOBAL BEHAVIOR

$P_T = K_T \Delta_T$ $\qquad \Delta_T = \Delta_1 = \Delta_2 = \Delta$ (STIFF COLLECTOR)

$P_1 = K_1 \Delta$

$P_2 = K_2 \Delta$

$$P_T = P_1 + P_2 = K_1\Delta + K_2\Delta$$
$$= (K_1 + K_2)\Delta$$
$$= K_T \Delta$$

$\therefore \underline{K_T = K_1 + K_2}$

$$\frac{P_1}{P_T} = \frac{K_1 \Delta}{K_T \Delta} = \frac{K_1}{K_T} = \frac{K_1}{K_1 + K_2}$$

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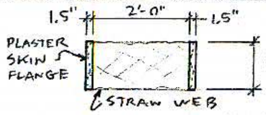
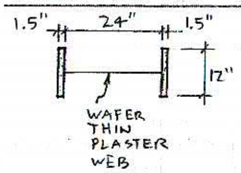
PRIMARY SYSTEM



$E_{SKIN} = 3{,}000{,}000$ psi $\quad E_{STRAW} = 130$ psi $\quad G_{STRAW} = 50$ psi

(8' TALL WALL) $L = 8'$

TRANSFORMED SECTION



TRANSFORMED WIDTH $b = 12''(130/3000000) = .00052''$ (NEGLECT)

$$\Delta_P = \underset{\text{(SHEAR)}}{\frac{PL}{4A_{WEB}G_{STRAW}}} + \underset{\text{(FLEXURE)}}{\frac{PL^3}{48E_{SKIN}I_T}} \qquad I_T = \frac{12(27)^3}{12} - \frac{12(24)^3}{12} = 5859 \text{ in}^4$$

$$= P\frac{8(12)}{4(12)24(50)} + P\frac{8^3(1728)}{48(3000000)5859}$$

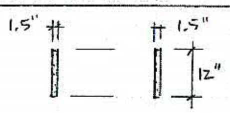
$$= .001667P + .000001P$$

(NEGLECT FLEXURE)

$P_P = K_P \Delta_P$

$\Delta_P = .00167 P_P \qquad K_P = 1/.00167 = 600^{\#/''}$

SECONDARY SYSTEM



2 SKINS BEND INDIVIDUALLY

$$I_{SKIN} = \frac{12(1.5)^3}{12} = 3.375 \text{ in}^4$$

$$\Delta_S = \underset{\text{2 SKINS}}{\left(\tfrac{1}{2}\right)} \frac{PL^3}{48E_{SKIN}I_{SKIN}} = \left(\tfrac{1}{2}\right)P\frac{8^3(1728)}{48(3000000)3.375} = .0009102P$$

$P_S = K_S \Delta_S$

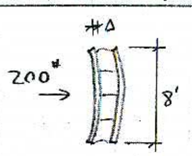
$\Delta_S = .0009102 P_S \qquad K_S = 1/.0009102 = 1099^{\#/''}$

SUPERPOSE 2 SYSTEMS TO FORM TOTAL SYSTEM

$K_T = K_P + K_S = 600^{\#/''} + 1099^{\#/''} = 1699^{\#/''}$

$K_P/K_T = 600/1699 = .353 \qquad K_S/K_T = 1099/1699 = .647$

TEST WALL W/ $P = 200^{\#}$ APPORTION LOAD TO 2 SYSTEMS ACCORDING TO STIFFNESS



$P_P = \left(\frac{K_P}{K_T}\right)P_T = .353(200^{\#}) = 70.6^{\#} \qquad \Delta = .00167(70.6) = .118''$

$P_S = \left(\frac{K_S}{K_T}\right)P_T = .647(200^{\#}) = 129.4^{\#} \qquad \Delta = .0009102(129.4) = .118''$

CHECK BENDING STRESS OF SKIN

P_S (LOAD ON SKINS) $= 129.4^{\#}$
LOAD ON SINGLE SKIN $P_{SS} = (.5)129.4^{\#} = 64.7^{\#}$

$M = P_{SS}L/4 = 64.7(8)12/4 = 1552.8^{\#''}$

$$f_b = \frac{Mc}{I} = \frac{1552.8^{\#''}(.75'')}{3.375 \text{ in}^4} = \underline{345} \text{ psi}$$

STRENGTH OF HARD SKIN (ASSUME CEMENT-PLASTER)

$f'_c = 2000$ psi MODULUS OF RUPTURE $f_{MOR} = 7.5\sqrt{f'_c} = 7.5\sqrt{2000} = \underline{335}$ psi

$\therefore$ AT TEST LOAD OF $200^{\#}$ $f_b = 345$ psi $\approx f_{MOR} = 335$ psi

(THIS IS FLEXURALLY EQUIVALENT TO $W = 200/4 = 50^{\#/'}$ - OR A WIND LOAD OF 50psf)

AFTER THE HARD SKIN CRACKS, THE GLOBAL SYSTEM TAKES ALL LOAD

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C

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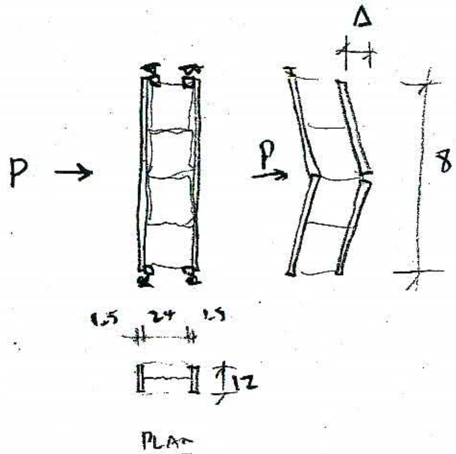
JOB NO. 5/13 SH

NO. 3 OF 3

BY KED DATE 4/16/01

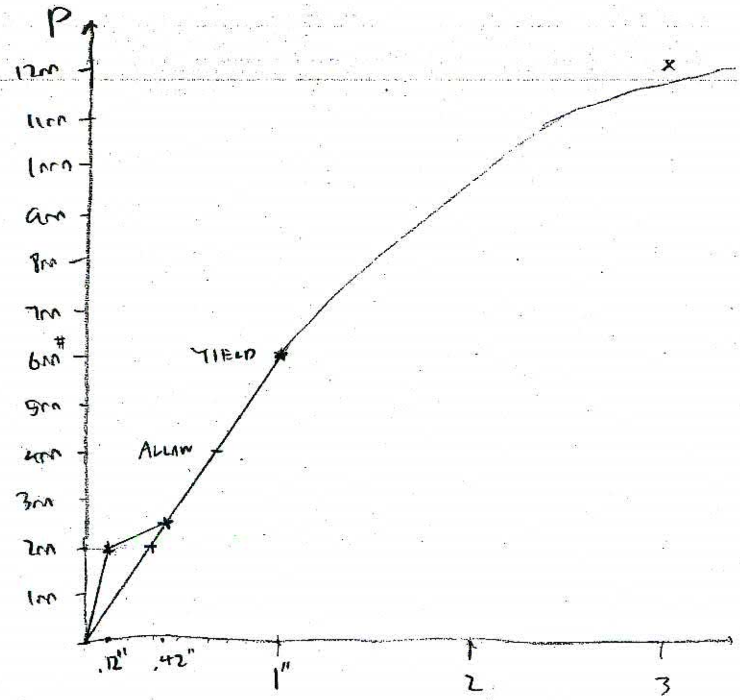
Fig 1 (Bigger) 12" SECTION OF WALL



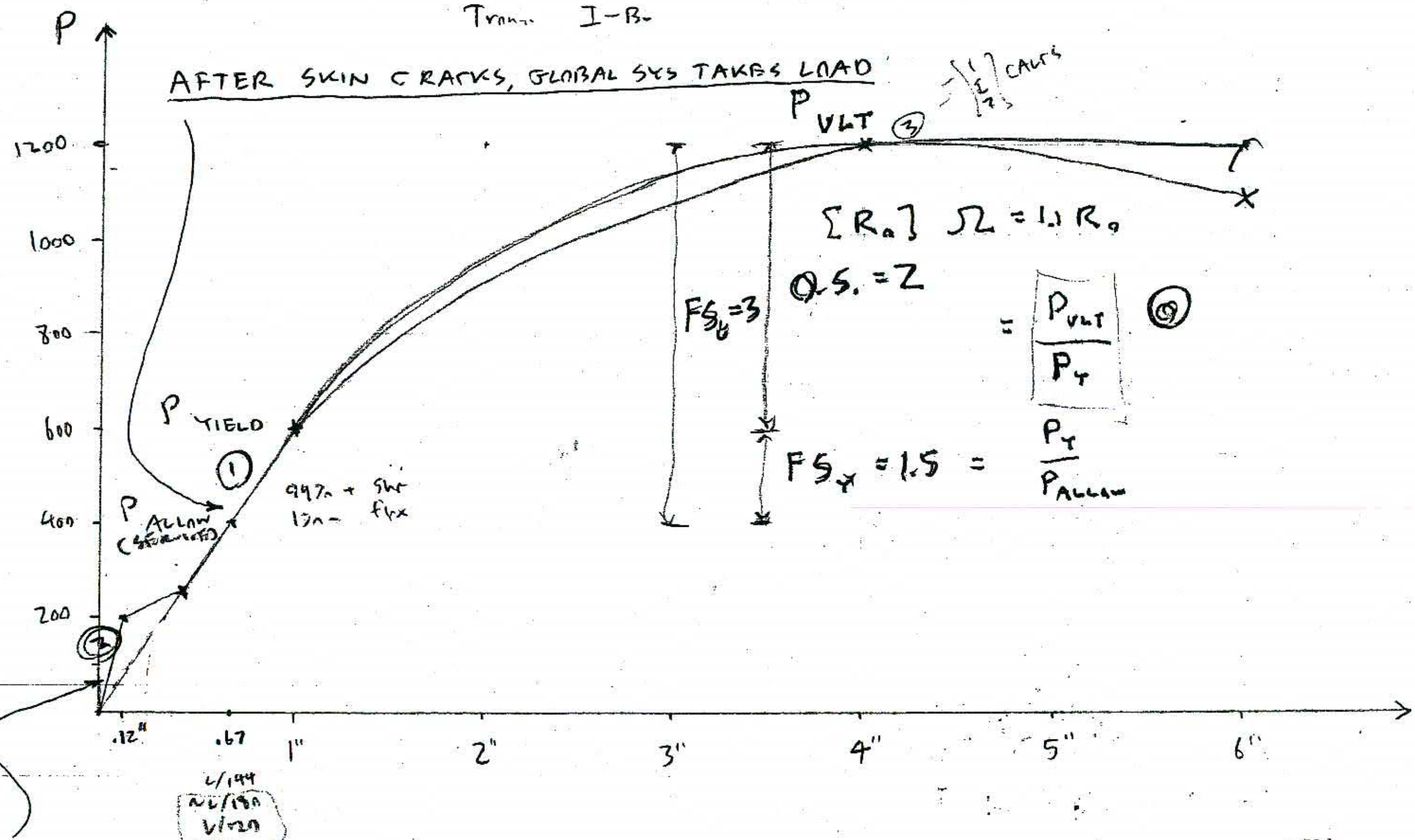
$E_{cp} =$

$E_{sm} =$

$t_{wm} =$



Basic Overview — Sandwich Panel — Composite Trans. I-Be
Hinges Simplified



HIGH INITIAL STIFFNESS (AT P = 200#, Δ = .118", SKIN CRACKS) .666 =

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SUPPLEMENTAL STRUCTURAL CALCULATIONS

IN SUPPORT OF

ICC CODE CHANGE
CHAPTER 24
STRAWBALE CONSTRUCTION

OUT-OF-PLANE LOADS
(PROPOSED CODE SECTIONS 2405.11 & 2405.12)

PAGES 91 THROUGH 103 OF "DESIGN OF STRAW BALE BUILDINGS" BY BRUCE KING

GREEN BUILDING PRESS, 2006

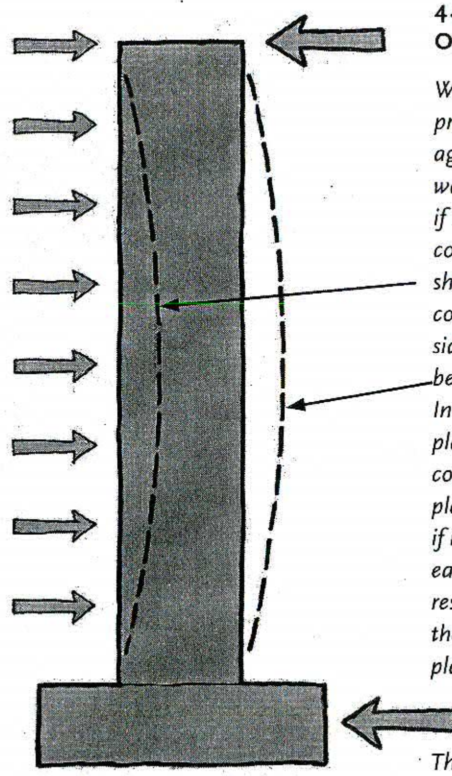
4.3 "OUT-OF-PLANE LOAD" WITH KEVIN DONAHUE

4.3 Out-of-Plane Load

with Kevin Donahue

4.3.1 Introduction

Forces applied perpendicular to a surface are called *out-of-plane* loads. Below grade, they are the pressure of earth, liquid, or both against basement and retaining walls. Above grade, the most common is simple wind pressure, which can cause many problems for structures as a whole, but very rarely for the walls themselves unless they are particularly tall or poorly supported. Although the pressures caused by wind are in fact anything but simple, varying and reversing both with specific location and with time, engineers designing residential-scale structures typically treat wind loads as uniform pressure applied towards (pressure) or away from (suction) the surface of a wall or roof.



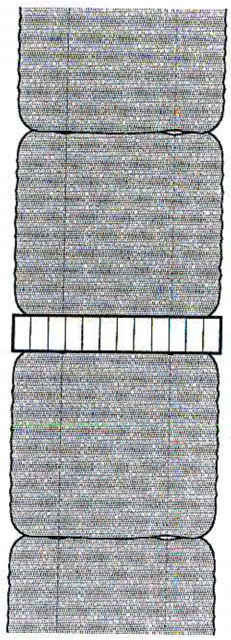
4.3A
OUT-OF-PLANE LOAD

When wind or other pressures are applied against the surface of a wall, it will bend, even if not noticeably. The concave side tries to get shorter because it is in compression; the convex side tries to get longer because it is in tension. In a straw bale wall, the plaster must resist the compression, and the plaster reinforcing, even if it is only straw fiber in earthen plaster, must resist tension forces above the nominal amount that plasters can withstand.

The supports at top and bottom must be able to resist the horizontal reaction force.

In the case of plastered straw bale walls, wind loads are demonstrably non-problematic, as shown in laboratory tests [endnotes 2, 3, 4, and 5] and in field experience – on at least two occasions, unplastered and unbraced straw bale walls withstood hurricane-force winds without distress. The remaining issue in designing for such pressure forces, then, is to model wall behavior as a basis for design of very tall or large wall panels. That discussion will comprise the bulk of this section.

There are other types of out-of-plane forces, however, that are both rare and far more destructive. In the extreme are the effects of bomb blasts or errant vehicles driven into the side of a building, for which there is no economical way to design with *any* building material. If you want a bunker, build a bunker, with three or ten feet of reinforced concrete all around as has been done for decades. If, however, you only want a reasonably safe structure – one that meets minimum requirements of your local building codes and can withstand the range of foreseeable environmental forces to be expected in its lifetime, then forget about the bombs and runaway trucks. Some events are both infinitesimally unlikely and dramatically costly to protect against. If a big meteor strikes your home, you can only hope that it hits the far end, away from your family, and you can make a lot of money selling your story to Hollywood.



4.3B Horizontal girt

A common method of adding horizontal (out-of-plane) support to a tall wall is to set a glulam beam (with a depth equal to the bale thickness) between two courses at mid-height. The girt must have slotted supports at its ends to allow some vertical movement as the bales are stacked, and should be attached to plaster mesh just before plastering.

Less extreme but more common are two quite different types of dynamic loading: the effects of objects *carried* by extreme winds, and the shaking effects of earthquakes. The former is recognized as the main problem for buildings in tornado and hurricane zones, for which tests have been developed and carried out on all manner of wall systems (including plastered straw bale walls; see section 4.3.7 on *projectile loads*).

As discussed in section 4.1, buildings in the proximity of known active earthquake faults will get mildly or even rudely shaken up a lot during their lifetime, and might very easily get whacked by The Big One. At the very least, you must allow for that in design and construction – a process somewhat like the wind pressure analysis you would or should do anyway.

To date most engineers have introduced horizontal girts (figure 4.3B) or some other type of additional support when the wall became taller than anything with which they were intuitively comfortable. The need remains, then, to explain how the various materials in the wall assembly work together in order to to lay out a design methodology for larger and taller walls. That is what follows.

4.3.2 Two-Phased Wall Behavior Under Out-of-Plane Load

Some early and relatively crude out-of-plane load tests on plastered straw bale walls, as well as recent and more sophisticated ones, began with a horizontal plaster crack appearing at or near the mid-height bale course on the tension face. This led many to conclude that the walls were more or less exhibiting classic bending behavior for stress-skin panels. Their conclusion was correct – up to that point of first cracking. However, like a concrete beam transitioning from the uncracked to the cracked mode, the behavior of the wall changes substantially with the appearance of that first crack in the tension face. In residential-scale walls, the first crack has always appeared well above design or service loads, but would nonetheless be of concern in two cases:

1) If the wall is underdesigned and experiences this cracking in extreme wind or a moderate earthquake, there would be no structural failure, but the wall could now be vulnerable to water intrusion, leading to longer-term moisture problems in the core (this is another reason for a good roof overhang.)
2) When (not if) the crack appears during major seismic shaking, it will compromise the wall's ability to carry in-plane loads. Emphasis here is on "compromise," not "lose"; the plaster's bond to the straw, along with the reinforcing mesh, will hold the skin together so that even a full-depth crack can transmit some shear via friction – similar to a phenomenon witnessed in seismic tests of adobe construction.

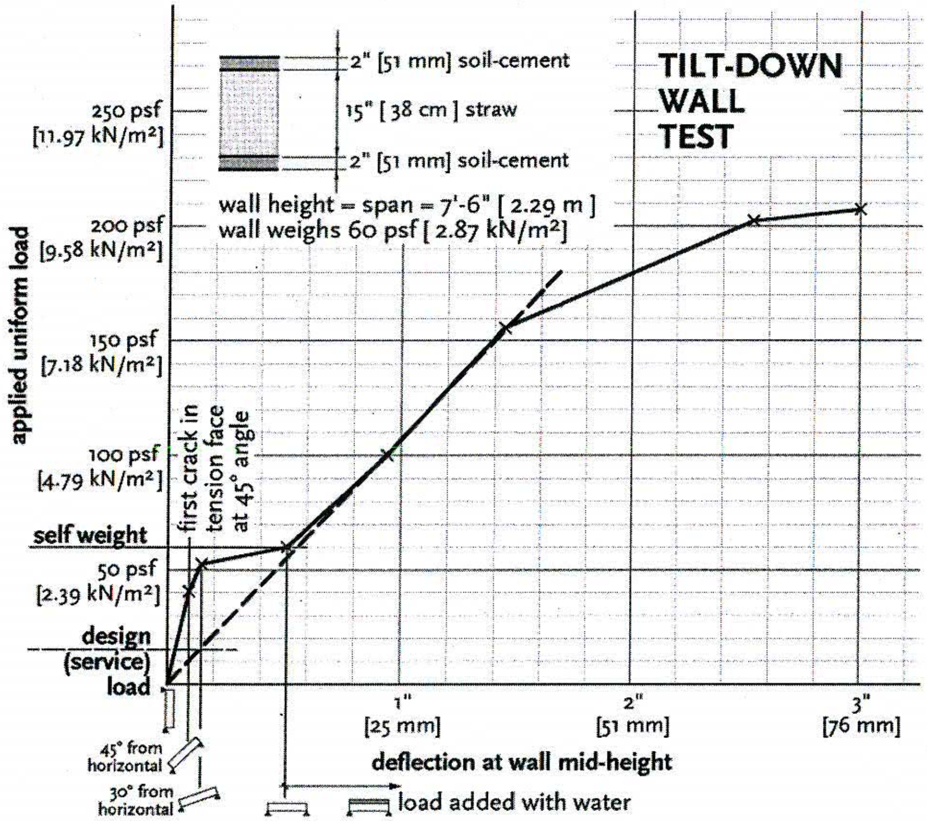
In section 4.1 we found that a plastered straw bale wall is in many ways like a stress-skin panel and can be modeled as a *sliced transformed section*. That model is conservative in that it assumes no frictional shear between bales and treats the course joints as slots in the web of the transformed section. The friction factor between bales laid flat, μ, has been measured[1] to be about 0.63, but the normal force between bales can vary widely from project to project, with time, and even within a single wall; the conservatism seems well-warranted. The sliced transformed section model also intimates that shear behavior will govern over bending, and out-of-plane tests to date have consistently shown evidence of exactly that: shear distortion of the bales dominates wall behavior.

Kevin Donahue conducted and studied several tests with the hope of identifying behavior and establishing a design methodology. All of those tests were conducted on wall specimens about eight feet [2.4M] high, a typical residential scale, and none showed anything like failure until they were loaded far beyond service loads.

In the first test[2], a 7.5 foot high [2.3 M] wall was built and plastered both sides with a two-inch [50 mm] sprayed soil-cement known as *Pisé* [A](a "semi-soft skin" plaster in the parlance of chapter 3, like a low-strength concrete), reinforced with 2" [5.1 cm] x 2" x 16 gauge welded wire mesh. The wall was then slowly rotated

FOOTNOTE

A
"Pisé" is both the historic french word for rammed earth, and an acronym coined by David Easton for Pneumatically-Applied Stabilized Earth.



The wall was built, plastered, and cured upright, then carefully rotated on hinged supports to the horizontal, measuring deflections at 45 and 30 degrees from horizontal. After it was flat, load was added with water in a plastic-lined box framed over the flat wall.

The dashed line represents the behavior of a wall following a pure shear deflection, given by the formula $\Delta_{shear} = \dfrac{wH^2}{8G_cA_c}$, *in which w is the uniform load, H is the height,* A_c *is the area of the core (straw), and* G_c *is the Shear Modulus given by* $G_c = E_c/(2(1+\nu))$ *where* ν (nu) *is Poisson's ratio, which has been measured for bales at 0.35. For an* E_c *of 130 psi+,* G_c *is then about 50 psi. (*ν *has only been measured once, for bales loaded flat, so the extrapolation to a* G_c *value for the bales is admittedly tenuous but does lead to calculations that well match test results.*

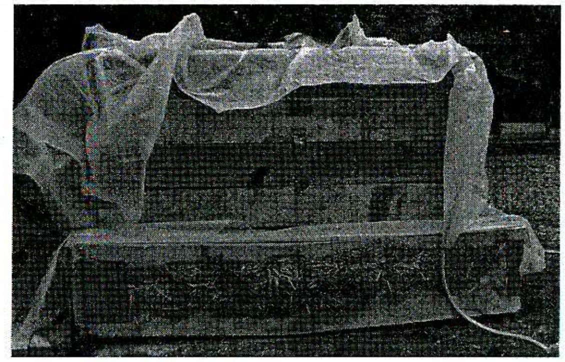
4.3C
Two-phased wall behavior under out-of-plane load

on hinged supports to a horizontal position, at which point it had an out-of-plane load of 1g (100% of self-weight), and had deflected 0.56 inches [14 mm]. Substantial additional load was then added by building a plastic-lined frame on top and adding water in measured increments. A detailed illustration of that test, showing the two-phased behavior just described, is given in figure 4.3C.

4.3.3 Pre-Cracking Wall Behavior Under Out-of-Plane Load

A complete analysis of most beams of most materials requires checking both bending and shear stresses, and deflections. As a practical matter, except for very short, heavily-loaded beams, shear is often a minor consideration, and most beams (and walls under out-of-plane load) are checked only for bending loads. As it turns out, with plastered straw bale walls the situation is different, as evidenced by every test to date. The straw bales, being relatively very soft, distort under load from rectangles to parallelograms within the wall, while the bonded sections of plaster try to remain rectangular; shear is by far the dominant force in both stress distribution and deflection.

In the Tilt-Down test, the first crack appeared in the tension face when the wall had been rotated down to 45 degrees from horizontal – a 42 psf [2 kN/m^2] load perpendicular to the surface – and the wall had deflected 0.09" [2 mm]. Up to that point, the plaster skins tried to act as shallow beams in bending to resist the load, as follows:



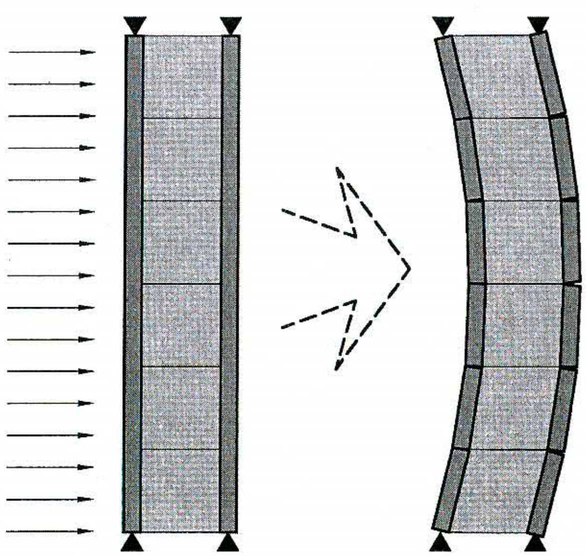
4.3D

The tilt-down wall in horizontal position with water-filled box on top for adding load.

For a 12-inch-wide vertical strip of wall:

- the moment on the wall is $M = wl^2/8 = 42(7.5^2)/8 = 295$ ft-lbs, or 3544 in-lbs
- the section modulus of the two-inch-thick plaster skins is $S = bh^2/6 = 12(2^2)/6 = 8\ in^3$ (two skins = 16 in^3)
- the bending stress at the point of cracking is $M/S = 3544/16 = 222$ psi
- check against predicted $MOR = 1.6\sqrt{f'c} = 1.6\sqrt{1200}$ (allowable stress design) = 55 psi

Given the level of precision (or imprecision) of the structural model, plaster thickness, and material strength, this is a very satisfying correlation; the plaster skins tried to resist the load as beams, and cracked when the tension reached the plaster's predicted capacity of four to five times the allowable stress. Further evidence of



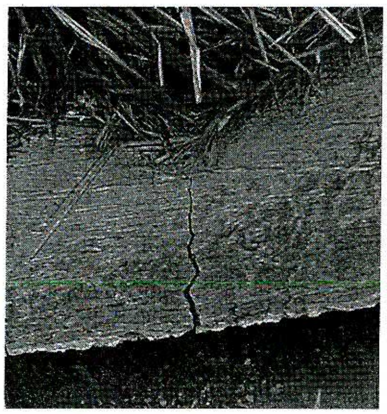
4.3E

When loaded out of plane, the soft straw bales distort from rectangles to parallelograms, forcing cracks to appear along course joints.

This behavior is consistent with every test done to date, but had been incorrectly interpreted to be due to sliding of the bales relative to one another.

Until the first crack appears, then, the load is resisted largely by the two plaster skins acting like wide, shallow beams – not very strong, but nonetheless stiffer than the global structural assembly. The first cracks occur when the tension face exceeds its tensile capacity – the Modulus of Rupture (MOR).

this behavior is the shape of the cracks; were there pure stress-skin-panel-type behavior, the cracks would evenly split the tension-side plaster skin and would be uniform in thickness. Such was not the case, as can be seen clearly in figure 4.3F



4.3F BENDING CRACK IN TENSION FACE PLASTER

The crack is wide at the bottom (the face of extreme tension), occurs at a bale coursing joint, and narrows to nothing within the thickness of the plaster, indicating that the plaster skin was trying to act as a very shallow beam and quickly reached failure (though at a load far above design load). This indicates that the wall has created a "hinge" point that shifts its behavior from bending-dominated to a complex combination of shear in the bales, and tension/compression/bending in the skins, as will be modeled below. The 2" x 2" 16 gauge mesh in the plaster is now fully engaged, similar (but not the same) to the way steel reinforcing is more fully engaged when a concrete beam cracks.

4.3.4 Post-Cracking Wall Behavior Under Out-of-Plane Load

Donahue modeled the cracked section, as shown in figures 4.3C and 4.3G, and developed a seven-step design methodology by which any wall, of any size and material makeup, can be analyzed. Following is a depiction of the method, using as an example a one foot wide [30 cm] strip of an eight foot high [2.4 M] wall subject to a 100 psf load, which "becomes" an 800 pound point load at mid-height. The reactions at points 1, 2, 5 and 6 are each 800/4 = 200 pounds. Use also the material properties used in the tilt-down wall test depicted above:

2 inch thick soil-cement plaster ("Pisé")

f'_c = 1200 psi +/- (based on cube tests)
Modulus of Elasticity $E_s = 57{,}000\sqrt{f'_c} \cong 1{,}975{,}000$ psi [13,617,000 kN/M²]
(Allowable Stress Method (ASD))

2" x 2" x 16 gauge welded wire mesh

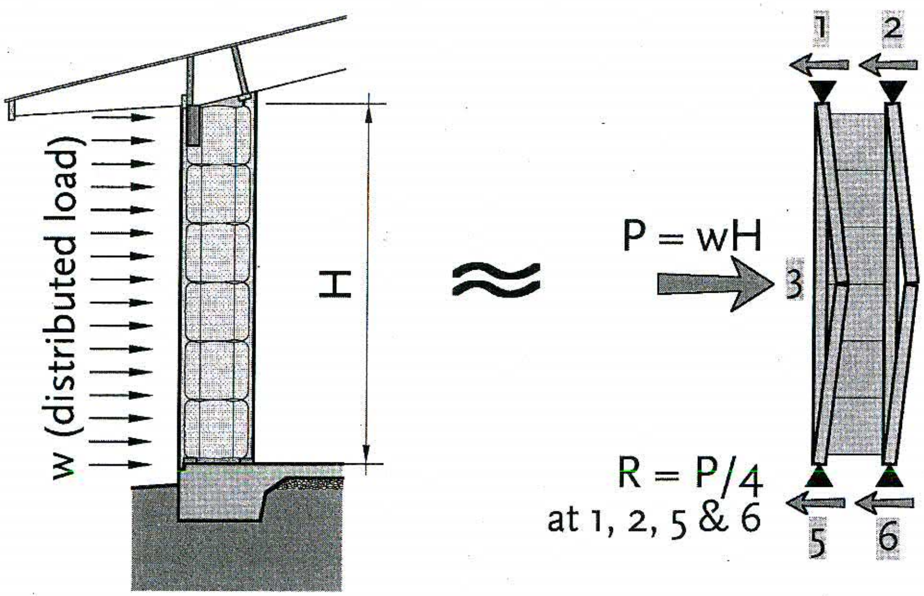
$A_s = 6 \times (.031^2 \times \pi) = 0.018$ in² / ft. [11.6 mm²] per linear foot [30 cm] of mesh
F_y = 60,000 psi [413,685 kN/M²]

16 gauge galvanized staples with 1³/₄" legs, 4 inches oc

National Evaluation Report NER 272 gives an allowable pullout force of 32 lbs. [142 N] each (which incorporates a safety factor of 5, i.e., average failure load was 5 x 32 = 160 lbs [712 N]). The allowable load per foot is, then, 3 x 32 = 96 pounds per foot [1401 N/m].

rice straw bales

E_c (core, i.e., straw) = 130 psi + [871 kPa]+



4.3G
DESIGN MODEL FOR POST-CRACKING WALL BEHAVIOR

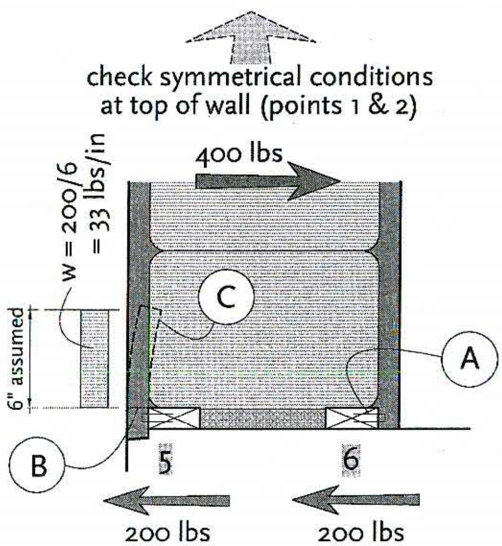
Notes:

1) Any engineer will look at this and note the factual error of basic static analysis: P does *not* equal wH. To get an equivalent bending moment on the wall, P would be wH/2. However, in the shear-dominated behavior of the post-cracking mode – the subject at hand – a P of wH gives the same reaction loads at points 1, 2, 5 and 6, and thus is the more representative approximation for analysis. This approximation, though apparently crude, fits very well with the data from all the tests.

2) In this model no vertical load is applied, i.e., the wall is assumed to be non-bearing. If the reader is analyzing a load-bearing wall, the vertical loads can be added to the model above, applied at points 1 and 2, reacted at points 5 and 6, and the static analysis as depicted in the following pages carried through. However, since tests have shown that residential scale walls eight or nine feet high [2.4-2.6 M] of just about any construction can easily resist conventional demand loads, the underlying assumption here is that a design is necessary because the wall is substantially higher than nine feet – a condition that for several reasons invariably calls for a post and beam structure with no vertical load (by definition) on the bale walls.

3) If the wall is part of a post-and-beam system, as is assumed in note 2, then there will be support on all four sides of the wall panel; that is, the vertical posts in the wall will also resist lateral load and brace the straw bale wall panel on its sides. Discounting that effect, substantial as it may be, renders this analysis conservative.

4.3.4a Example Seven-Step Calculation (steps listed as A through G)



4.3H

A) Check staple tension at points 2 and 6

R (demand) = 200 lbs, and the listed capacity is 96 lbs (but no staple failure was observed in tests at this load level; $\Longrightarrow$ the staple tension factor of safety is less than that which generated the NER report allowable value but is still serviceable. Bearing at points 1 and 5 may be carrying more than half the load.

B) Check plaster shear at points 1 and 5

R (demand) = 200 lbs, and the shear stress on the 2" x 12" cross section just above the sill plate is:

200 x 1.5 / (2x12) = 12.5 psi

Plaster (soil-cement) is like a weak concrete;

$\Longrightarrow V_{allow} = 1.1\sqrt{f'c} = 1.1\sqrt{1200} = 38$ psi (ASD) *OK*

C) Check plaster in bending at points 1, 2, 5, and 6

R (demand) = 200 lbs; treat as a uniform load of 33 pounds per inch on a section of plaster six inches high above a "fixed base" at the top of the sill plate. (The six-inch height is a judgment call, a somewhat arbitrary number agreed upon by the three engineers involved with developing these criteria: Kevin Donahue, David Mar and Bruce King. Six inches is slightly less than half the height of most bales and probably the most that a section of plaster can "span" in cantilever.)

section modulus of plaster (neglecting reinforcing) is $S = bh^2/6 = 12 \times 2^2/6 = 8$ in^3

Moment on section $M = w|^2/2 = 33 \times 6^2/2 = 594$ in-lb.

bending stress $f_b = M/S = 594/8 = 74$ psi (conservatively neglects effect of mesh reinforcing)

Modulus of Rupture MOR $\approx 1.6\sqrt{f'c} = 1.6\sqrt{1200} = 55$psi

$\Longrightarrow F_b \approx 55 < 74$ *OK w/implicit safety factor of 5 used in ASD values*

D) Check tension in bales

Shown as a freebody diagram, each bale in the wall will look like figure 4.3J, and will distort into a parallelogram as shown. Mar[3] postulated a "compression strut" acting within a wall assembly that must be activated to carry shear loads; likewise, a tension and compression strut will activate within the bale cross-section to resist – or try to resist – the distortion shown. Bale dimensions vary, but for the purposes of this exercise assume a 15 inch high by 12 inch wide section; the tension (and compression) stress is then: $(400\sqrt{2}) / (12x15\sqrt{2})$ 2.2 psi [15.2 kPa] – the demand load. This translates to 317 psf [15.2 kN/m²], and though we have no measurement (and can't imagine how to make one) of internal tensile strength, this is more than we would expect the bale to be capable of.

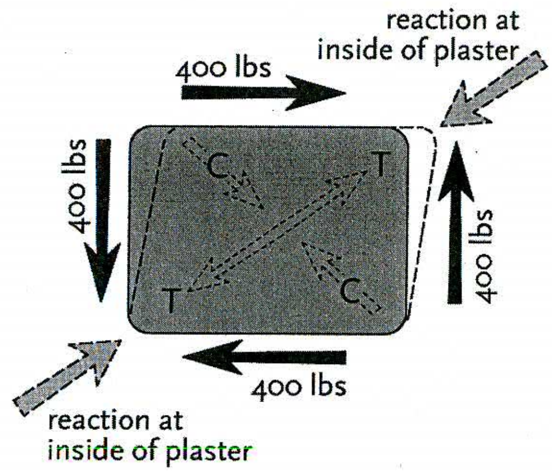


FIGURE 4.3J

The tensile strength of straw is known to be higher than most softwoods, but that has little relevance here. The internal tensile strength in a bale is a function of fiber lengths, and the degree to which they "grab" each other (be it by friction, mechanical interlock, or something else) determines how strong the bale is. That being the case, the orthogonal compression stress would be sqeezing the fibers together in a way that would increase frictional bond and therefore tensile capacity.

In the preceding load-deflection diagram, the wall had deflected almost an inch [24 mm] under a (very high) 100 psf [4.8 kN/m²] load, so we know the bales were distorted, and that by extrapolation 2 psi +/- is higher than the bale's tensile yield point. And *that* means that the distortion of the bale must be resisted by the diagonal reactions shown at the plaster in combination with whatever tensile capacity the bales in fact have.

The skins are both acting as the facings of a stress-skin panel and holding the wall assembly together, and shear is flowing from bales to plaster to bales and so on. These point loads at the bale corners – the bale course joints – explain the horizontal cracks that have appeared in every out-of-plane test to date. It may well be that there is no slippage at all between bales, as had been thought before; the diagonal point reactions shown in the freebody diagram would be enough to crack most plasters. The "slotted transformed section" model (in section 4.1) probably still applies, but appears to be conservative.

This discussion also suggests the purpose, and perhaps need, for wire or string ties that connect the mesh through the wall at bale courses. A tie would in theory carry the tension force, relieving both the bale and the plaster skin from having to do so. (We say "in theory" because the test results to date do not necessarily confirm this; as is ever the case, more research is needed to clarify this behavior.) Wall ties will certainly strengthen and stiffen a wall against out-of-plane loading, and should be considered essential in conditions of very high seismic risk and/or in very tall walls.

The diagram also shows that there is shear (bond) stress at the straw-plaster interface. For a fifteen-inch-high bale, that stress is $f_v = 400/(12x15) = 2.2$ psi [15 kN/m^2] (as per basic mechanics, the same as the previously-computed tension stress). As with the tensile bond to be discussed in the section that follows, we have no measurement of what the failure stress may be, but have seen no overt slippage in tests to date. Furthermore, innumerable reports from the field portray plaster and straw as being *very* difficult to separate when circumstances require it. The implication is that this shear bond is not a limiting link in the complex mechanism that is a plastered straw bale wall.

E) Check tensile bond between plaster and straw at points 1 and 5

We checked in steps B and C that the plaster skin bearing against the sill plate can resist the 200 pound reaction, but we must also look at the tendency of the bales to pull away from the plaster at those same locations. Using the same model as for step C, the tensile bond stress would be distributed over a six inch x twelve inch area: $f_t \approx 200/(6x12) = 2.8$ psi [19 kN/m^2]. Once again, we have no measured values against which to compare this demand load, but wall tests to date show no evidence of this being a problem.

The shear and tensile bond between straw and plaster both depend primarily on three things:

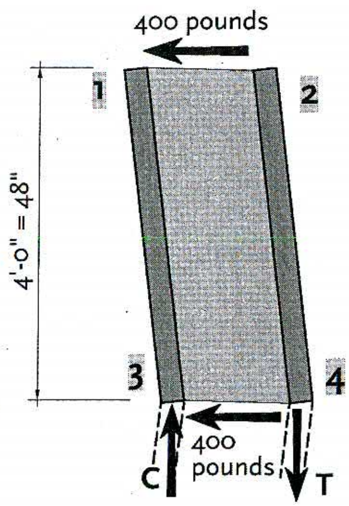
1) the average length of straw fibers (the longer the better),

2) the number of fibers per unit area that engage the plaster enough to have "development length," and

3) the amount those same embedded fibers engage, or "grab," the mass of straw in the bale.

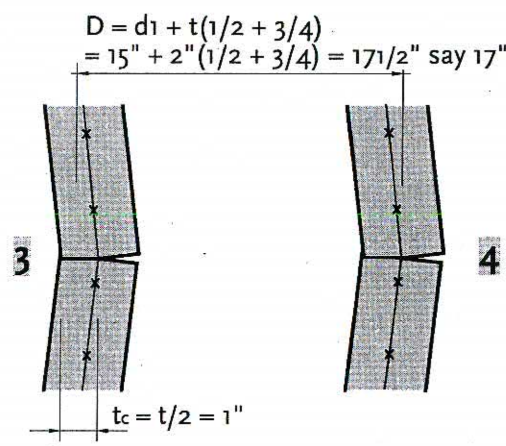
All of this, though hugely difficult to quantify, emphatically points to the value of using bales with the longest possible fibers (see chapter 1, *Straw and Bales*), and the need to work the first coat of plaster well into the straw (see chapter 3, *Plaster and Reinforcing*).

F) Check mesh tension at point 4



4.3K

A freebody diagram of the top half of the wall, showing the shear, tension, and compression forces at the mid-height hinge.



4.3L

Detail of the hinge joints at wall mid-height (points 3 and 4)

The tension and compression forces T and C are easily calculated to be 400 x 48" / 17" = 1129 pounds [5 kN]. The mesh area was calculated to be 0.018 in^2/foot, so the mesh tensile stress f_t = 1129/0.018 = 63 ksi [434,370 kN/m^2]. This is roughly equal to the ultimate strength of most galvanized wires, more than enough to cause some yielding.

G) Check compression on plaster at point 3

Assuming that half the plaster thickness remains engaged, as shown, then the plaster stress is f_c = 1129/(1"x12") = 94 psi [648 kN/m^2]. Compared to the allowable compressive stress of 0.33 f'_c, or 400 psi, this is well within allowable limits.

4.3.5 Comments

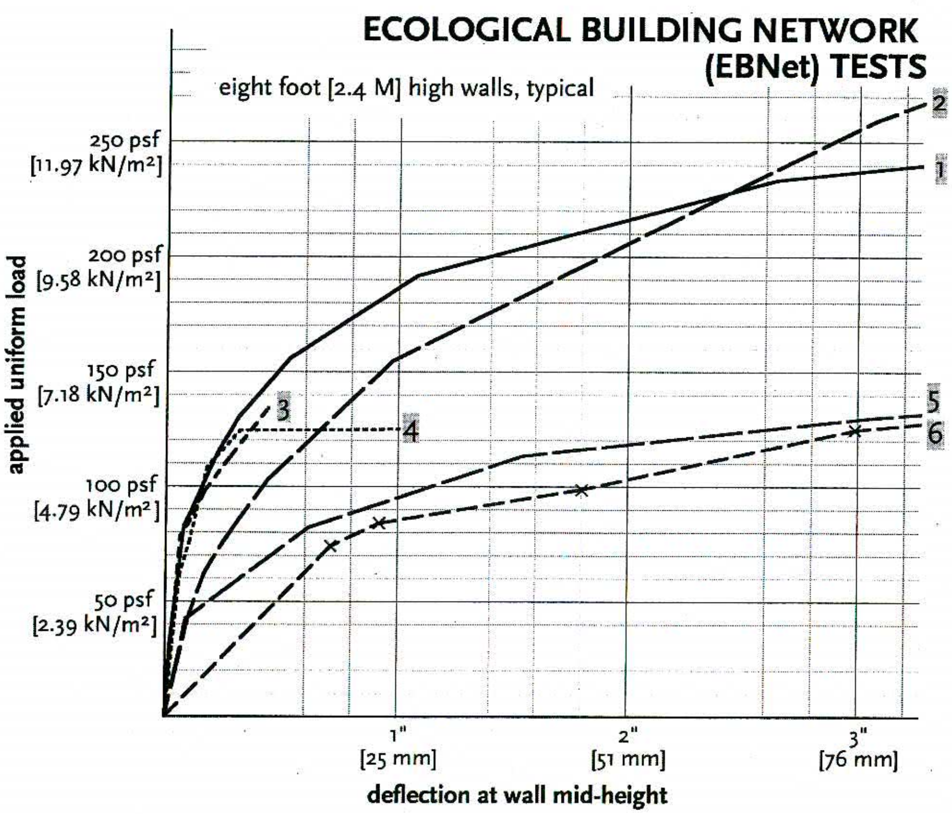
In the foregoing example we analyzed a wall almost exactly like the "tilt-down" test wall depicted in the preceding load-deflection diagram, the difference being that we used a slightly taller wall (8 feet high, not 7.5 feet). More importantly, the applied load of 100 psf [4.8 kN/m²] is far higher than any but the most rare and extreme wind or earthquake demands on a residential-scale structure anywhere in the world. Walls of that size (roughly 8 feet/2.4 M high), plastered with just about anything, seem to be abundantly stiff and robust; even unplastered walls of that scale have passed without problem through hurricane-level winds.

We have tentatively presented a design methodology that treats the wall as a simple span from foundation to roof or floor above (and conservatively ignores the bracing effect of vertical posts within the wall, where they occur). In doing so we have necessarily speculated, since the whole wall assembly, as should by now be obvious, is an ungainly combination of bales (about whose properties we know comparatively little), plaster (about which we may know a lot, or little, circum-

4.3M
Ecological building network out-of-plane tests, December, 2003

All walls are eight foot high stacks of 16" x 23" x 48" bales (23" perpendicular to load) with plaster, mesh, and attachments as shown (the same on both sides of the wall). Load was applied in a semi-cyclical way, in that the bladder that applied the load (see figure 4.3N) could be inflated up to the point of applying a desired pressure but would not sustain it; there was load relaxation between each reading.

(The full report can be downloaded at www.ecobuildnetwork.org)



stances depending), staples, and mesh (about which we generally know more). Still, this is educated speculation, well-matched to test results,[2,3,4,5] and articulates a much more detailed and logical model of wall behavior than has previously been described.

(See also section 4.5 for general comments about design recommendations.)

4.3.6 Summary of Recent Tests

Wall 1:
Plaster: one-inch stucco applied in two coats
Plaster reinforcement: 2 x 2 x 14 gauge welded wire fabric stucco mesh
Top and bottom connection: 16 gauge, 7/16"crown x 1.75" leg staples @ 2"

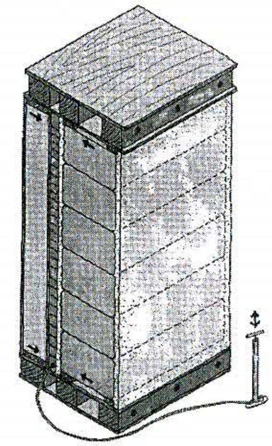
Wall 2:
Plaster: one-inch stucco applied in two coats
Plaster reinforcement: 17 ga x 1.5" hexagonal woven wire lath
Top and bottom connection: 16 ga 1/2"crown x 1.25" leg staples @ 6"

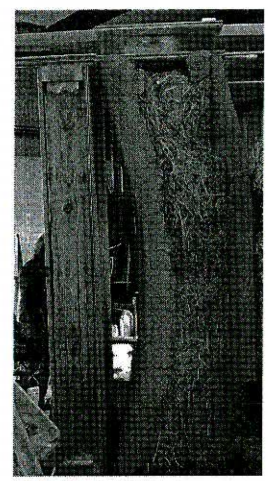
Wall 3:
Plaster: one-inch stucco applied in two coats
Plaster reinforcement: 1% 1.5" Xorex steel fibers by volume (13 lbs per side)
Xorex specs: deformed 0.045" x 1.5" 120 ksi fibers

Wall 4:
Plaster: one-inch stucco applied in two coats
Plaster reinforcement: 0.8% 2" Xorex steel fibers by volume (10.5 lbs per side)
Xorex specs: deformed 0.045" x 2.0" 120 ksi fibers

Wall 5:
Plaster: two-inch earth plaster applied in one coat
Plaster reinforcement: 2 x 2 x 0.047" Cintoflex C plastic mesh
Through-ties: two loops (of 2) baling twine spaced @ 24" each course (seven courses) tied to 5/8" x 4' horizontal bamboo dowels outside mesh both sides

Wall 6:
Plaster: two-inch earth plaster applied in one coat
Plaster reinforcement: 2 x 2 x 0.047" Cintoflex C plastic mesh
Thru ties: two loops (of 2) baling twine spaced at 24" above the second and fourth courses
(third points) tied to 5/8" x 8' vertical bamboo dowels outside mesh both sides





4.3N
OUT-OF-PLANE TEST APPARATUS, WALL IN FAILURE-LEVEL DEFLECTION, AND FIRST CRACK

illustration by David Mar
photos courtesy of Kevin Donahue