STRUCTURAL ENGINEERING HANDBOOK

STRUCTURAL ANALYSIS OF ICS 3-D WALL PANELS

PREPARED BY

THE CONSULTING ENGINEERS GROUP, INC.
MT. PROSPECT, IL

FEBRUARY, 1991

STATE OF ILLINOIS
LESLIE D.
MARTIN
81-346[illegible]
LICENSED STRUCTURAL ENGINEER

Publication 1-C, September 28, 1992

COMPANY NAME________________________ PROJ MGR______________

PHONE__________________ PROJECT NAME__________________________

STARTUP DATE___________ ESTIMATED COMPLETION DATE_____________

ARCHITECT OF RECORD________________________ PHONE____________

STRUCTURAL ENGINEER OF RECORD___________________ PH__________

NOTES:____

____

____

____

____

ICS CONTACTS:

NAME: ______________________________ PHONE:___________________

NAME: ______________________________ PHONE:___________________

TOOLS & MATERIALS REQUIRED TO CONSTRUCT WITH ICS 3-D PANEL SYSTEM

1. Pneumatic ICS Fastener Tool (Example:Spenax Model SC-643 - ¾")
2. Fastening Rings or Clips)Example: Spenax Hog Rings #16G-110)
3. Reciprocating Saw (Hand held variable speed preferred with minimum 8" metal cutting blade with 20-24 teeth per inch)
4. 7-¼" Circular Saw (With metal cutting blade)
5. Wire Loop Ties and Hand Twister Tool (To secure panels to reinforcing steel)
6. Basic Hand Tools For Construction (Must include level-minimum 4 ft. to true-up panels)
7. Screed Material (Can be stucco screeds, plastic pipe or wire guides)
8. Concrete Pump For Pneumatic Application Of Concrete (Can be shotcrete, gunite or mortar pump)
9. Finishing Tools (Trowels, sponges, darbys, or any other hand tool to give desired concrete finish)
10. Air Compressor (Minimum size 6.6 CFM)

STRUCTURAL ANALYSIS OF 3-D WALL PANELS

I. DESCRIPTION

The 3-D wall panel consists of a three-dimensional welded wire space frame integrated with a polystyrene insulation core. This reinforcement/insulation module is placed in position and wythes of concrete or mortar are applied to both sides. This is shown conceptually in Fig. 1.

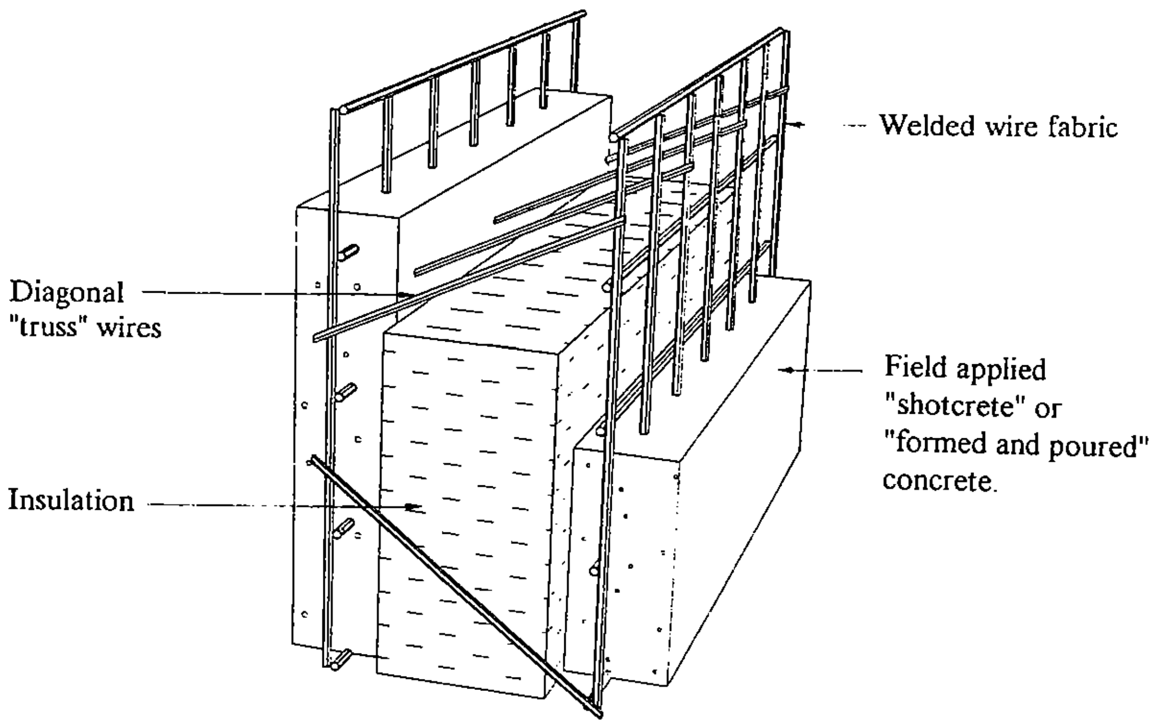


Fig. 1. The 3-D Wall Panel

The 3-D wall panel receives its strength and rigidity by the diagonal cross wires welded to the welded wire fabric on each side. This produces truss behavior which is very rigid and provides adequate shear transfer for composite behavior.

The reinforcement/insulation module (RIM) is shop fabricated with highly automated equipment. This insures consistent dimensional control and high quality welding. The standard 3-D panel is shown in Fig. 2. Other thicknesses of insulation and concrete, as well as slightly different wire gauges, are also available. Information on these other configurations can be supplied by your ICS 3-D Panel representative.

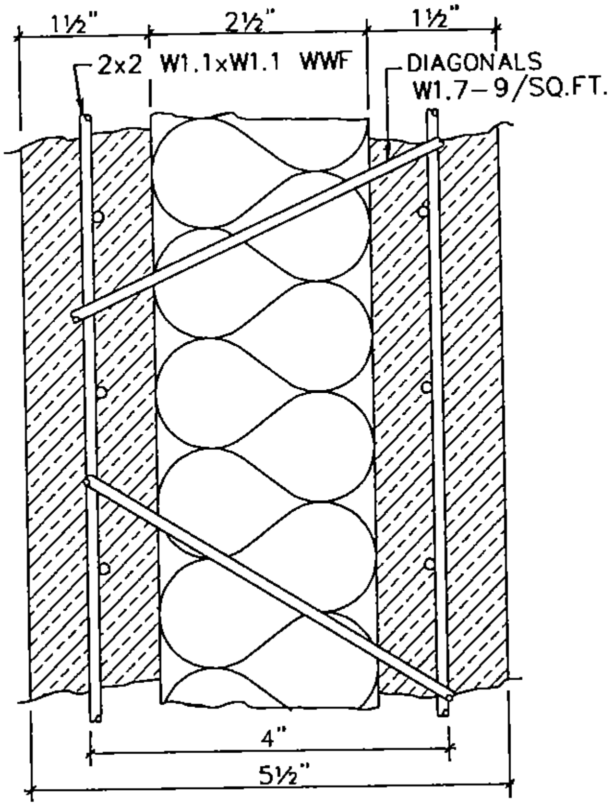


Fig. 2. Standard 3-D Section

II. TEST VERIFICATION

In 1985, an extensive test program of panels constructed of nearly identical materials and construction techniques was performed at the Graz Technical University in Graz, Austria. This program consisted of both flexural (beam) tests and compression (column) tests.

In 1990, a number of flexural (beam) tests were performed at the ICS 3-D Panel Works, Inc. manufacturing facilities in Brunswick, Georgia. The test specimens were loaded in a manner to best simulate a uniform load on the panel. The tests were designed and witnessed by independent licensed professional engineers.

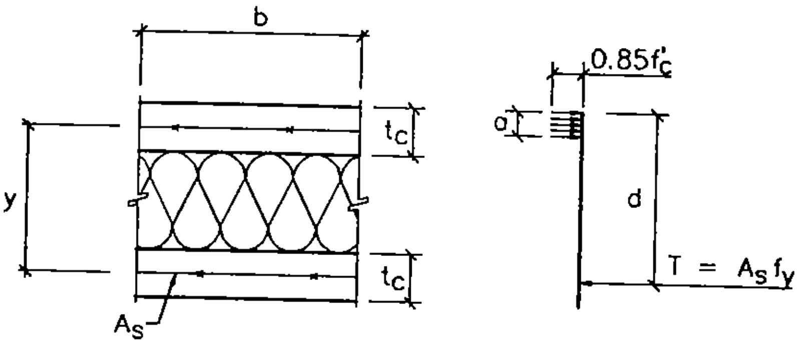
Also in 1990, a series of ten tests of three specimens each were tested as load-bearing wall panels, generally following procedures outlined in ASTM E-72. These tests were performed by a nationally recognized testing laboratory.

The results of the tests verify that design can be safely accomplished by standard procedures. Copies of the complete test reports and verification calculations are available by contacting your ICS 3-D panel representative.

III. DESIGN OF 3-D PANELS

The testing programs described above verified the 3-D panels can be designed by standard strength design methods specified in the American Concrete Institute "Building Code Requirements for Reinforced Concrete (ACI 318)," and described in textbooks on reinforced concrete design. Briefly:

Flexure:



$$\phi\ M_n = \phi\ A_s f_y\ (d - a/2)$$

$$\text{Where } a = \frac{A_s f_y}{0.85 f'_c b} \leq t_c$$

$$\phi = 0.9$$

Shear:

$$\phi\ V_n = \phi\ bd\ (0.5\sqrt{f'_c})$$

$$\phi = 0.85$$

Note: This is less than the shear allowed by ACI 318, but is based on test data.

Deflection:

Test results show 3-D wall panels do not behave as fully composite sections. For deflection calculations under wind loads, it is recommended that the effective moment of inertia be conservatively assumed to be 1/5 of the calculated gross moment of inertia:

$$I_g = \frac{Ay^2}{2} = \frac{bt_c y^2}{2}$$

For Standard 3-D panels, per foot of width:

$b = 12\ in. \quad t_c = 1.5\ in. \quad y = 4\ in.$

$$I_g = \frac{12(1.5)(4)^2}{2} = 144\ in.^4$$

$$I_e = I_g/5 = 28.8\ in.^4$$

Compression and Compression with Bending:

Tests have shown that 3-D panels can be designed for compression by using interaction curves constructed by standard strain compatibility. An interaction curve for the standard 3-D panel is shown in Fig. 3. slenderness effects can be accounted for by using the moment magnification procedure described in ACI 318-89. Section 10.11.5, as follows:

$$\delta = \frac{1}{1 - \dfrac{P_u}{\phi P_c}}$$

For $k = 1$

$$P_c = \frac{\pi^2 EI}{(l_u)^2}$$

$$EI = \frac{E_c I_g/5}{1 + \beta_d}$$

Note: Comparison with tests show that I_g *rather than* I_e *may be used for calculation.*

$$E_c = 57000\sqrt{f'_c}$$

$$\phi = 0.7$$

In-Plane Shear:

When 3-D panels are used as shear walls to resist lateral loads, the walls are designed in accordance with ACI 318-89, Section 11.10. The total thickness of the wall, h, in this case is the sum of the two concrete wythes.

$$V_u \leq \phi \, (V_c + V_s)$$

$$V_c = 2 \sqrt{f'_c} \; hd$$

$$d = 0.8 l_w$$

$$V_s = \frac{A_s f_y d}{s}$$

$$\phi = 0.85$$

A. NON-BEARING WALLS

When 3-D panels are used to clad buildings framed with other materials, they are usually designed to resist wind forces as flexural members. The following example illustrates such a use:

Example 1: Standard 3-D panels, 20 feet high, form the external walls of an industrial building framed with steel rigid frames. The local code requires 18 psf wind pressure.

$$M = \frac{wl^2}{8} = \frac{18\,(20)^2}{8} = 900 \; ft\text{-}lb/ft = 0.9 \; ft\text{-}kips/ft$$

$$Load\ factor\ (ACI\ 318) = 1.3$$

$$M_u = 1.3\ (0.9) = 1.17\ ft\text{-}kips/ft$$

$$V_u = 1.3\ (18)\ (20/2) = 234\ lb/ft$$

Capacity:

$$Conservatively\ assume\ f'_c = 2500\ psi\ and\ f_y = 56{,}000\ psi$$

For 12 *in. width*:
A_s = *W*1.1 *wires* = 6(0.011) = 0.066 *in.*^{2}/*ft*

Total panel thickness = 5.5 *in.*
WWF located 3/4 *in. each face*

$$d = 5.5 - 0.75 = 4.75\ in.$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.066(56000)}{0.85(2500)(12)} = 0.14\,in.$$

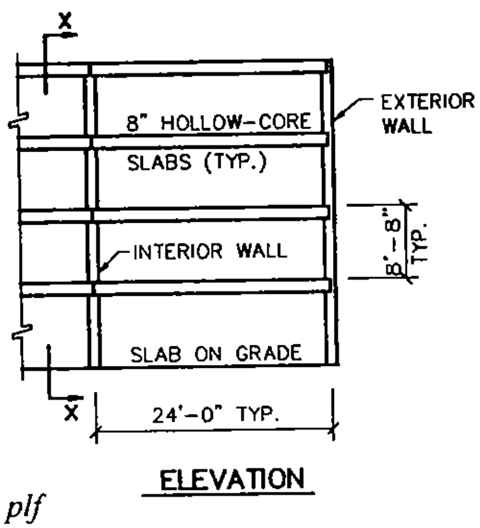
$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(0.066)(56)(4.75 - \frac{0.14}{2}) = 15.6\,in.-kips/ft$$

$$\phi V_n = \phi bd\ (0.5\sqrt{f'SUBc} = 0.85(12)(4.75)\ (0.5\sqrt{2500}) = 1211\,lb > 234\ lb$$

$$Deflection = \frac{5wl^4}{384 E_c I_e} = \frac{5(18)(20)^4(1728)}{384(57000\ \sqrt{2500})(28.8)} = 0.79\ in. = l/304 < l/240$$

B. BEARING WALLS

Example 2: A four-story residential struture uses standard 3-D panels for both interior and exterior walls. The loading is as follows:



<u>Dead Load</u> - 60 psf

<u>Live Load</u> -

Typical - 40 *psf*
Corridor - 80 *psf*
Roof - 80 *psf*

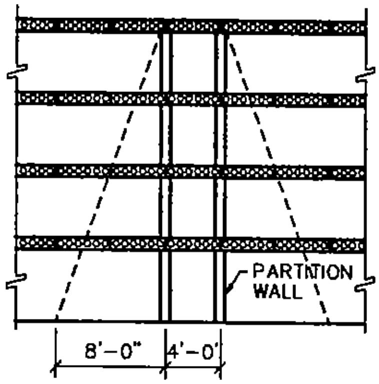
<u>Wind Load</u> - 20 psf

a. <u>Interior Walls</u>

<u>Dead Load</u>

Floors & Roof = 60 *psf x* 24 = 1440 *plf x* 4 = 5760 *plf*

$$Corridor = \frac{60 \ x \ 2}{8} \ x \ 24 = 360 \ x \ 4 = 1400 \ plf$$



DISTRIBUTION OF CORRIDOR LOADS

SECTION X–X

$$Wall = 3.75 \ psf \ x \ 8 \ x \ 4 = 1200 \ plf$$

$$Total = 5760 + 1440 + 1200 = 8400 \ plf$$

<u>LiveLoad</u>

$$Floor - 40 \ psf \ x \ 24 = 960 \ plf/flr.$$

$$Corridor - \frac{80 \ x \ 2}{8} \ x \ 24 = 480 \ plf/flr.$$

$$Roof - 25 \ psf \ x \ 24 = 600 \ plf$$

$$Total = (960 + 480) \ x \ 3 + 600 = 4920 \ plf$$

$$Total \ Factored \ Max. \ Load = 1.4(8.40) + 1.7(4.92) = 20.1 \ kips = P_u$$

$$Assume \ minimum \ eccentricity = 1 \ in. \ M_u = \frac{20.1(1)}{12} = 1.68 \ ft-kips$$

<u>Slenderness Effects</u>

$$f'_c = 2500 \ psi$$

$$E_c = 57000 \sqrt{f'_c} = 57000\sqrt{2500} = 2.85 \ x \ 10^6 \ psi \ or \ 2850 \ ksi$$

$$I_g = \frac{Ay^2}{2} = \frac{1.5(12)(4)^2}{2} = 144 \ in^4/ft$$

$$\beta_d = \frac{\textit{Dead Load Moment}}{\textit{Total Load Moment}} = \frac{\textit{Factored Dead Load}}{\textit{Factored Total Load}} = \frac{8.4(1.4)}{20.1} = 0.59$$

$$EI = \frac{E_c I_g / 5}{1 + \beta_d} = \frac{2850(144)/5}{1.59} = 51{,}623$$

$$P_c = \frac{\pi^2 EI}{(l_u)^2} = \frac{\pi^2(51{,}623)}{(8 \ x \ 12)^2} = 55.3 \ kips$$

$$\delta = \frac{1}{1 - \dfrac{P_u}{\phi P_c}} = \frac{1}{1 - \dfrac{20.1}{0.7(55.3)}} = 2.10$$

$$M_u = 2.10(1.68) = 3.53 \ ft\text{-}kips$$

From Interaction curve, Fig. 3, wall design is OK.

Fig. 4, which includes slenderness effects, can also be used with $P_u = 20.1 \ kips, \ and \ e = 1 \ in.$

b. Exterior Walls

Dead Load

$Floor \ \& \ Roof = 60 \ psf \ x \ 12 \ x \ 4 = 2880 \ plf$
$Wall = 37.5 \ x \ 8.67 \ x \ 4 = \underline{1300}$

LiveLoad

Conservatively, consider wall at corridor.

$Floor \ at \ Corridor = 80 \ psf \ x \ 12 \ x \ 3 = 2880 \ plf$
$Roof = 25 \ psf \ x \ 12 = \underline{300} \ plf$
$3180 \ plf$

$$P_u = 1.4(4.18) + 1.7(3.18) = 11.3 \ kips$$

$$M_u = \frac{11.3(1)}{12} = 0.94\ ft\text{-}kips$$

Wind moment - continuous wall

$$M = \frac{0.020(8)^2}{10} = 0.13\ ft\text{-}kips$$

For combined D + L + W

$= 0.75[1.4D + 1.7L + 1.7W]$

$$P_u = 0.75(11.3) = 8.5\ kips$$

$$M_u = 0.75[0.94 + 1.7(0.13)] = 0.87\ ft\text{-}kips$$

Slenderness:

$$\beta_d = \frac{Factored\ Dead\ Load}{Factored\ Total\ Load} = \frac{1.4(4.18)}{11.3} = \frac{5.85}{11.3} = 0.52$$

(*Note*: β_d *does not apply to wind load moments*)

$$EI = \frac{E_c I_g /5}{1+\beta_d} = \frac{2850(144)/5}{1.52} = 54{,}000$$

$$P_c = \frac{\pi^2 EI}{l_u^2} = \frac{\pi^2(54{,}000)}{(8\ x\ 12)^2} = 57.8\ kips$$

$$\delta = \frac{1}{1 - \dfrac{11.3}{0.7(57.8)}} = 1.39$$

$$D + L \quad M_u = 0.94(1.39) = 1.30\ ft\text{-}kips$$

$$D + L + W \quad M_u = 0.87(1.39) = 1.21\ ft\text{-}kips$$

See Figs. 3 and 4

Example 3: The wall panels of Example 1 support a steel framed roof which spans 60 ft. The roof dead load is 15 psf, and the live load is 25 psf.

$$D.L.\ to\ 3\text{-}D\ Panel = (60/2)(15) = 450\ plf$$

$$L.L.\ to\ 3\text{-}D\ Panel = (60/2)(25) = 750\ plf$$

$$Wall\ weight = 37.5\ psf \times 20 = 750\ plf$$

$$P_u = 1.4(450 + 750) + 1.7(750) = 2955\ lb/ft$$

Assume roof load bearing is 2 in. eccentric

$$M\ (Dead) = 1.4(450)2 = 1260\ in\text{-}lb$$

$$M\ (Total) = [1.4(450) + 1.7(750)](2) = 3810\ in\text{-}lb$$

$$\beta_d = \frac{1.4(450)}{2955} = 0.21$$

$$E_c = 57000\sqrt{f'_c} = 57000\sqrt{2500} = 2.85 \times 10^6\ or\ 2850\ ksi$$

$$I_g = 144\ in^4/ft\ (See\ example\ 2)$$

$$EI = \frac{E_c I_g/5}{1+\beta_d} = \frac{2850(144)/5}{1.21} = 67{,}835$$

$$P_c = \frac{\pi^2 EI}{l_u^2} = \frac{\pi^2(67{,}835)}{[20(12)]^2} = 11.6\ kips/ft$$

$$\delta = \frac{1}{1 - \frac{P_u}{\phi P_c}} = \frac{1}{1 - \frac{2.95}{0.7(11.6)}} = 1.57$$

For D + L

$$P_u = 2.95^k$$

$$M_u = \frac{3810}{12000} \; x \; 1.57 = 0.16 \; ft\text{-}kips/ft$$

D + L + W

$$P_u = 0.75(2.95) = 2.21 \; kips$$

$$\delta = \frac{1}{1 - \dfrac{2.21}{0.7(11.6)}} = 1.37$$

$$M_u = 0.75 \left[\frac{3810}{12000}(1.37) + 1.7 \; x \; 0.9\right] = 1.47 \; ft\text{-}kips/ft$$

See Fig. 3

Fig. 4 may also be used with

$$P_u = 2.21 \; kips$$

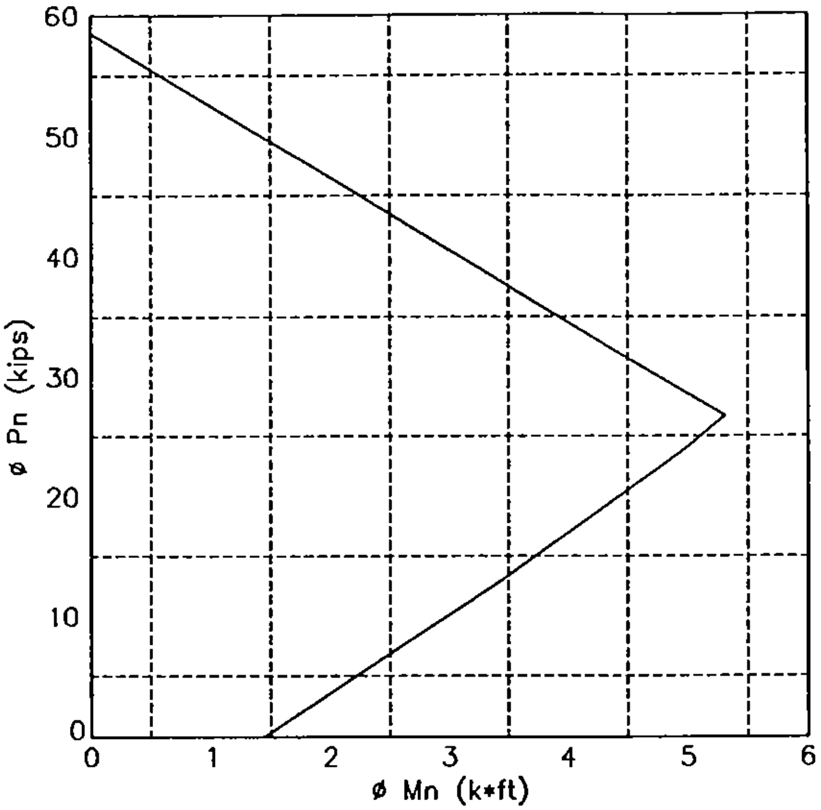
$$e = \frac{1.47 \; x \; 12}{2.21} = 8.0 \; in.$$

C. SHEAR WALLS

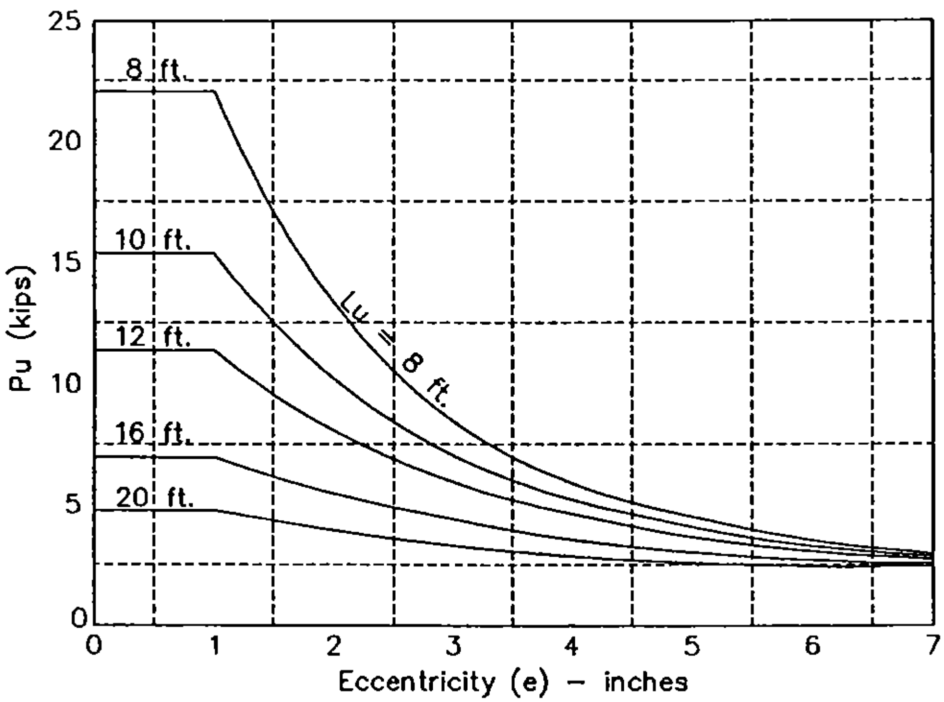
Example 4: The four-story building of Example 2 is assumed to be located in a seismically active region. The hollow-core floor is assumed to act as a diaphragm, distributing the seismic forces to the ICS 3-D bearing walls. By following the procedure in the applicable building code, it has been determined that the shear force in the plane of the wall is 60 kips (full width of building):

$$V_u = 60(1.3)(1.1) = 86 \; kips$$

$$h = 2(1.5) = 3 \; in.$$



Interaction curve
Standard ICS 3-D Wall Panel
f'c = 2500 psi (min) fy = 56 ksi
Figure 3



Interaction curve combined with slenderness
Standard ICS 3-D Wall Panel
f'c = 2500 psi (min) fy = 56 ksi β_d = 0.7
Figure 4

$$Wall\ on\ each\ side\ of\ column\ =\ 28\ ft$$

$$d = 0.8(2)(28) = 44.8\ ft = 538\ in.$$

$$V_c = 2\sqrt{f'_c}\ hd = \frac{2\sqrt{2500}}{1000}(3)(538) = 161.4\ kips$$

$$Each\ wire\ area = 0.011\ in^2\ spaced\ at\ 2''\ (each\ wythe)$$

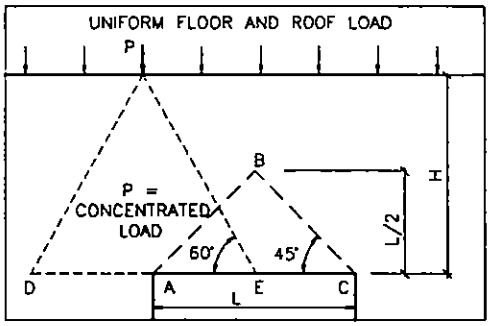
$$A_v = 2(0.011) = 0.022$$

$$V_s = \frac{A_v f_y d}{s} = \frac{0.022(56)(538)}{2} = 331.4\ kips$$

$$\phi V_n = \phi(V_c + V_s) = 0.85(161.4 + 331.4) = 419\ kips \rangle\ 86\ kips$$

D. WINDOW AND DOOR OPENINGS

In the design of ICS 3-D wall panels which span over window and door openings, recommendations of the brick industry[1] can be used as guides for determining loads:



In the figure above, the dead weight of the wall which must be supported over the opening is assumed as the weight of the triangle ABC. Arching action carries the weight of the wall and superimposed loads outside this triangle, provided that the wall above point B and to the sides of the opening is sufficient to provide resistance to the arching thrusts. For most conditions, 8 in. above and 24 in. on the sides is sufficient.

[1] Technical Notes on Brick Construction, No. 17H, Brick Institute of America, Reston, VA, July 1986.

If the uniform load is less than 8 in. above the opening, then the load that must be carried is that over the full width of the opening. In any case, only that portion of the wall weight within the triangle ABC must be considered.

Concentrated loads supported on the wall above the opening may be considered distributed over a width DE, and that portion over the opening (i.e. AE) included in the design.

Once the loads have been determined, the portion of the wall above the opening can be designed as a fixed-end beam by reinforced concrete strength design principles and the requirements of ACI 318. The following assumptions are recommended:

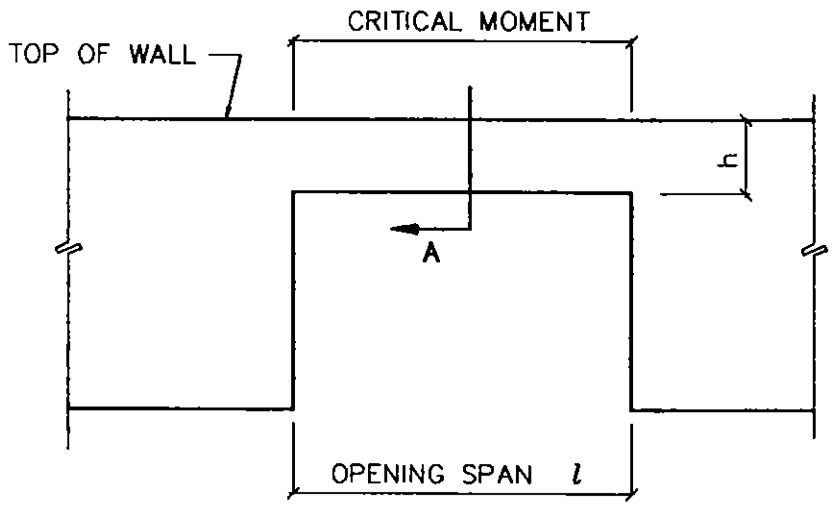
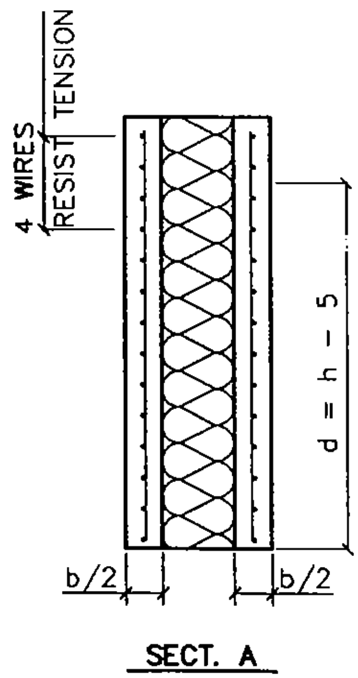


Fig. 5



SECT. A

A_s = *8 x Area of one wire*

For standard 3-D Panel:

$A_s = 8(0.0011) = 0.088\ in.^2$

$$a = \frac{A_s f_y}{0.85\, f'_c\, b}$$

$$\phi M_n = 0.9\, A_s f_y (d - a/2)$$

$$V_c = 2\sqrt{f'_c}\, bd$$

$$V_s = \frac{A_v f_y d}{s}$$

A_v = *area of vertical wires*
s = *spacing of vertical wires*

$$\phi V_n = \phi(V_c + V_s)$$

Note: In this direction, the diagonal wires do not transfer beam shear, so ϕ-factors and allowable strengths specified in ACI 318 may be used.

In the case illustrated above, the reinforcement percentage A_s/bd, is less than $200/f_y$, so the section must be designed for a moment 1/3 greater than that determined by analysis (ACI 318-89, Sect. 10.5.2). In some cases it may be advantageous to add reinforcing bars above the opening so that $A_s/bd \geq 200/f_y$, and the section can be designed for the calculated moment.

Example 5: The wall opening in Fig. 5 is 10′-0″, h = 24 in. Standard ICS 3-D panel, b = 3 in., A_s = 0.088 in^2, f_y = 56 ksi (minimum), f'_c = 2500 psi (minimum). Wall weight = 37 psf. There is a uniformly applied roof load at the top of the wall equal to 250 lb/ft dead load and 450 lb/ft live load.

In order to simplify the calculations, conservatively neglect the triangular loading pattern, but assume the critical section for shear is at d/2 from the edge of the opening.

$$\textit{Dead load} = [250 + 2(37)]1.4 = 454 \ \textit{lb/ft}$$

$$\textit{Live load} = 450(1.7) = 765 \ \textit{lb/ft}$$

$$W_u = 454 + 765 = 1219 \ \textit{lb/ft}$$

Try without additional reinforcement:

$$-M_u = \frac{W_u l^2}{12} = \frac{1.219(10)^2}{12}(1.33) = -13.51 \ \textit{ft-kips}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.088(56)}{0.85(2.5)(3)} = 0.77 in.$$

$$d = 24 - 5 = 19''$$

$$\phi M_n = 0.9 \ A_s f_y (d - a/2) = 0.9(0.088)(56)(19 - 0.77/2)/12 = 6.88 \ \textit{ft-kips}$$

Try adding 2 - #3, Grade 60 reinforcing bars 2 in. above opening. $A_{sb} = 2(0.11) = 0.22$ in². d - 24 - 2 = 22".

$$Reinf.\ ratio = \frac{A_s}{bd} = \frac{0.22}{3(22)} + \frac{0.088}{3(19)} = 0.0049$$

$$Minimum = 200/f_y = 200/56000 = 0.0036 < 0.0049$$

$$M_u = 13.5/1.33 = 10.16 > 6.88$$

Add 2 - #3, Grade 60 reinforcing bars 2 in. below the top of the wall. This balances the bars above the opening so that

$$M_n(from\ bars) = A_s f_y (d-d')\ (conservative)$$

$$\phi\ M_n = 0.9(0.22)(60)(22-2)/12 = 19.80\ ft\text{-}kips$$

$$Total\ \phi\ M_n = 19.80 + 6.88 = 26.68\ ft\text{-}kips > 10.16$$

Determine shear at d/2 = 0.79 ft from edge of opening (Note: This is conservative, since major reinforcement is the bars, d of 22 in. could be used).

$$V_u = 1.219(10/2 - 0.79) = 5.13\ kips$$

$$V_c = 2\sqrt{f'_c}\,bd = 2\frac{\sqrt{2500}}{1000}(3)(19) = 5.70\ kips$$

$$V_s = \frac{A_v f_y d}{s} = \frac{0.011(56)(19)}{2} = 5.85\ kips$$

$$\phi V_n = \phi(V_c + V_s) = 0.85(5.70 + 5.85) = 9.82\ kips > 5.13\ kips$$

E. ANCHORS TO FOUNDATION

ICS 3-D wall panels are normally anchored to cast-in-place concrete footings or foundation walls by reinforcing bars. These bars are embedded in both the foundation and the wall panel enough to develop the full tensile strength of the bar. A minimum of two bars should be used in full four foot wide panels, or partial panels greater than two feet wide. One anchor should be used in partial panels two feet or less in width.

The bars in the foundation can be either placed before casting the concrete or placed with chemical anchors after the foundation has hardened. Preplaced bars must be embedded in accordance with ACI 318-89 Sect 12.2 or 12.5. Chemically anchored bars should be of a manufacture approved by the applicable codes and placed in accordance with the manufacturer's instructions.

Bars must be placed very accurately in the direction perpendicular to the panel face, as they are required to project to the inside (insulation) side of the wire fabric layer, and still have enough cover to develop the full tensile strength of the bar. Bars should be no larger than #4, and #3 are preferred because of the cover requirements.

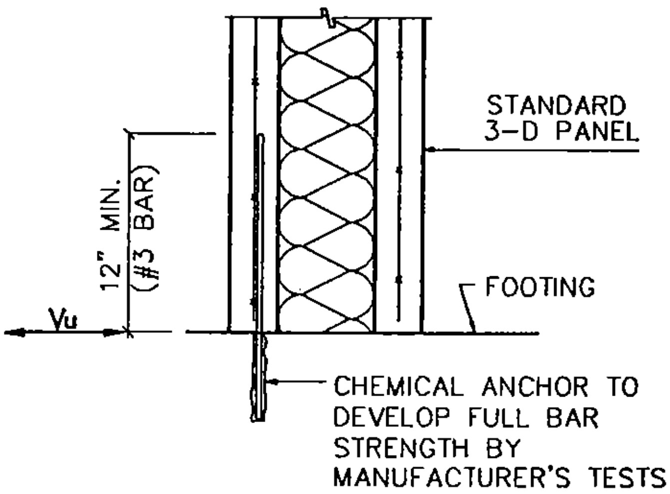
Design of the anchors for shear forces is by shear-friction in accordance with ACI 318-89 Sect. 11.7. The shear-friction coefficient, μ, should be 0.6.

Example 5: The connection below is at the base of a 20 ft high wall which must resist 18 psf of wind pressure or suction. Maximum spacing of bars is 2'-0":

$$V_u = 1.3(18)(20/2) = 234\ lb/ft$$

$$2(234) = 468\ lb$$

From ACI 318-89
Section 11.7.4.3



$$\mu = 0.6$$

$$V_n = A_{vf} f_y \mu$$

For a #3, Grade 60 reinforcing bar

$$V_n = 0.11(60)(0.6) = 3.96\ kips$$

$$\phi = 0.85$$

$$\phi V_n = 0.85(3960) = 3366\ lb > 468\ lb$$

Development length - *ACI* 318-89 *Sect.* 12.2.2

$$0.04\ A_b f_y / \sqrt{f'_c} = 0.04(0.11)(60{,}000)/\sqrt{2500} = 5.28''$$

Cover may be d_b *or less*

Sect. 12.2.3.2 = 2.0(5.28) = 10.56

Sect. 12.2.1 12 *in. minimum (use)*

SUPPLEMENT TO

STRUCTURAL ENGINEERING

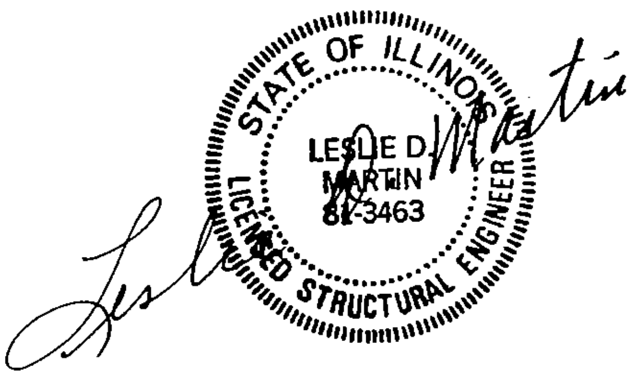
HANDBOOK

IV. ANALYSIS OF ICS 3-D WALLS WITH ONE FREE EDGE

PREPARED BY

THE CONSULTING ENGINEERS GROUP, INC.
MT. PROSPECT, IL

AUGUST, 1991



IV. ANALYSIS OF ICS 3-D WALLS WITH ONE FREE EDGE

In some applications of 3-D wall panels, the top edge is unsupported. Examples of these applications include fences, sound deflecting walls, and gable ends of houses or other buildings where the closure above the eave is wood or other material that does not provide support to the 3-D wall panel. These walls can be conservatively analyzed as simple or continuous beams spanning horizontally without support at top or bottom. Somewhat more precision, however, can be attained by modifying equations for flat plates given in the standard reference "Roark's Formulas for Stress and Strain" by Warren C. Young. Following are procedures and examples for using this approach. The derivation of tables is given in Section IV-B.

A. ANALYSIS USING TABLES

The coefficients in Table 1 are for flat plates simply supported on three sides and free on the fourth side. To determine the design moment, M_u, on a wall in ft-kips per ft of width, multiply the coefficient from Table 1 by the appropriate continuity coefficient shown in Fig. 6, the lateral load, usually wind, required by analysis or local building code in lb per sq ft and the appropriate strength design load factor, for example, 1.3 if the load is from wind. The procedure for calculating deflections is the same, except load factors are not used.

In Table 2, the load factor, 1.3, and the continuity coefficients from Fig. 6 are incorporated. Thus, merely multiply the coefficient by the required wind load.

The derivations of the tables are shown in Section IV B. The deflection coefficients are based on the standard ICS 3-D wall section shown in Fig. 2, page 2 of the Structural Engineering Handbook. The moment of inertia is the I_e as calculated on page 4:

$$I_e = 28.8 \text{ in}^4$$

and the modulus of elasticity is based on a concrete strength, $f'_c = 2500$ psi.

$$E_c = \frac{57000}{1000}\sqrt{f'_c} = 2850 \; ksi$$

Therefore EI = 28.8 (2850) = 82,080 kip-in.^{2}

If different concrete strengths or wall thicknesses are used, the coefficients can be modified by multiplying by:

TABLE 1: MODIFIED ROARK COEFFICIENTS									
	SPAN BETWEEN SUPPORTS (FT)								
	14	16	18	20	22	24	26	28	30
H (FT)	MOMENT COEFFICIENTS								
6	0.0101	0.0115	0.0130	0.0144	0.0158	0.0173	0.0187	0.0202	0.0216
8	0.0130	0.0154	0.0173	0.0192	0.0211	0.0230	0.0250	0.0269	0.0288
10	0.0157	0.0182	0.0211	0.0240	0.0264	0.0288	0.0312	0.0336	0.0360
12	0.0188	0.0215	0.0243	0.0276	0.0310	0.0346	0.0374	0.0403	0.0432
14	0.0219	0.0251	0.0283	0.0315	0.0350	0.0389	0.0429	0.0470	0.0504
16	0.0228	0.0286	0.0322	0.0359	0.0395	0.0432	0.0476	0.0521	0.0567
H (FT)	DEFLECTION COEFFICIENTS								
6	0.0046	0.0069	0.0098	0.0135	0.0179	0.0233	0.0296	0.0370	0.0455
8	0.0061	0.0092	0.0131	0.0180	0.0239	0.0310	0.0395	0.0493	0.0606
10	0.0075	0.0114	0.0163	0.0225	0.0299	0.0388	0.0493	0.0616	0.0758
12	0.0085	0.0132	0.0195	0.0268	0.0358	0.0466	0.0592	0.0739	0.0909
14	0.0094	0.0146	0.0216	0.0307	0.0416	0.0541	0.0689	0.0863	0.1061
16	0.0098	0.0161	0.0237	0.0336	0.0461	0.0617	0.0785	0.0983	0.1211

TABLE 2: MOMENT AND DEFLECTION COEFFICIENTS

CASE 1: 2-SPAN WALL CONTINUOUS

	SPAN BETWEEN SUPPORTS (FT)								
	14	16	18	20	22	24	26	28	30
H (FT)	MOMENT COEFFICIENTS								
6	0.0131	0.0150	0.0168	0.0187	0.0206	0.0225	0.0243	0.0262	0.0281
8	0.0169	0.0200	0.0225	0.0250	0.0275	0.0300	0.0324	0.0324	0.0374
10	0.0204	0.0237	0.0274	0.0312	0.0343	0.0374	0.0406	0.0437	0.0468
12	0.0244	0.0280	0.0316	0.0359	0.0403	0.0449	0.0487	0.0524	0.0562
14	0.0285	0.0326	0.0367	0.0409	0.0455	0.0505	0.0558	0.0612	0.0655
16	0.0297	0.0372	0.0419	0.0466	0.0514	0.0561	0.0618	0.0677	0.0737
H (FT)	DEFLECTION COEFFICIENTS								
6	0.0018	0.0028	0.0039	0.0054	0.0072	0.0093	0.0118	0.0148	0.0182
8	0.0025	0.0037	0.0052	0.0072	0.0096	0.0124	0.0158	0.0198	0.0243
10	0.0030	0.0046	0.0065	0.0090	0.0120	0.0155	0.0197	0.0246	0.0303
12	0.0034	0.0053	0.0078	0.0107	0.0143	0.0186	0.0237	0.0296	0.0364
14	0.0038	0.0059	0.0086	0.0123	0.0166	0.0216	0.0276	0.0345	0.0424
16	0.0039	0.0064	0.0095	0.0134	0.0184	0.0247	0.0314	0.0393	0.0484

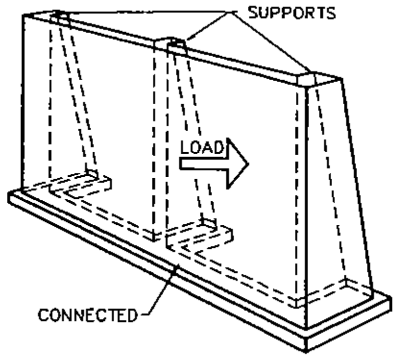
CASE 2: END SPAN OF 3 OR MORE SPANS

	SPAN BETWEEN SUPPORTS (FT)								
	14	16	18	20	22	24	26	28	30
H (FT)	MOMENT COEFFICIENTS								
6	0.0110	0.0126	0.0142	0.0157	0.0173	0.0189	0.0204	0.0220	0.0236
8	0.0142	0.0168	0.0189	0.0210	0.0231	0.0252	0.0273	0.0294	0.0314
10	0.0172	0.0199	0.0230	0.0262	0.0288	0.0314	0.0341	0.0367	0.0393
12	0.0205	0.0235	0.0265	0.0301	0.0339	0.0377	0.0409	0.0440	0.0472
14	0.0239	0.0274	0.0309	0.0343	0.0382	0.0424	0.0468	0.0514	0.0550
16	0.0249	0.0312	0.0352	0.0392	0.0431	0.0472	0.0519	0.0569	0.0619
H (FT)	DEFLECTION COEFFICIENTS								
6	0.0024	0.0036	0.0051	0.0070	0.0093	0.0121	0.0154	0.0192	0.0236
8	0.0032	0.0048	0.0068	0.0093	0.0124	0.0161	0.0205	0.0256	0.0315
10	0.0039	0.0059	0.0085	0.0117	0.0155	0.0202	0.0257	0.0320	0.0394
12	0.0044	0.0068	0.0101	0.0139	0.0186	0.0242	0.0308	0.0385	0.0473
14	0.0049	0.0076	0.0112	0.0160	0.0216	0.0281	0.0358	0.0449	0.0552
16	0.0051	0.0084	0.0123	0.0175	0.0240	0.0321	0.0408	0.0511	0.0630

TABLE 2 (CONTINUED): MOMENT AND DEFLECTION COEFFICIENTS CASE 3: INTERIOR SPAN OF 3 OR MORE SPANS SPAN BETWEEN SUPPORTS (FT)									
	14	16	18	20	22	24	26	28	30
H (FT)	MOMENT COEFFICIENTS								
6	0.0089	0.0102	0.0115	0.0127	0.0140	0.0153	0.0165	0.0178	0.0191
8	0.0115	0.0136	0.0153	0.0170	0.0187	0.0204	0.0221	0.0238	0.0255
10	0.0139	0.0161	0.0186	0.0212	0.0233	0.0255	0.0276	0.0297	0.0318
12	0.0166	0.0190	0.0215	0.0244	0.0274	0.0306	0.0331	0.0356	0.0382
14	0.0193	0.0222	0.0250	0.0278	0.0309	0.0344	0.0379	0.0416	0.0446
16	0.0202	0.0253	0.0285	0.0317	0.0349	0.0382	0.0420	0.0460	0.0501
H (FT)	DEFLECTION COEFFICIENTS								
6	0.0011	0.0017	0.0024	0.0032	0.0043	0.0056	0.0071	0.0089	0.0109
8	0.0015	0.0022	0.0031	0.0043	0.0057	0.0075	0.0095	0.0118	0.0146
10	0.0018	0.0027	0.0039	0.0054	0.0072	0.0093	0.0118	0.0148	0.0182
12	0.0020	0.0032	0.0047	0.0064	0.0086	0.0112	0.0142	0.0177	0.0218
14	0.0023	0.0035	0.0052	0.0074	0.0100	0.0130	0.0165	0.0207	0.0255
16	0.0024	0.0039	0.0057	0.0081	0.0111	0.0148	0.0188	0.0236	0.0291

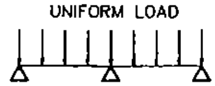
CASE 4: WALL CONTINUOUS WITH SUPPORTED CROSS WALLS
SPAN BETWEEN SUPPORTS (FT)

	14	16	18	20	22	24	26	28	30
H (FT)	MOMENT COEFFICIENTS								
6	0.0068	0.0078	0.0088	0.0097	0.0107	0.0117	0.0127	0.0136	0.0146
8	0.0088	0.0104	0.0117	0.0130	0.0143	0.0156	0.0169	0.0182	0.0195
10	0.0106	0.0123	0.0142	0.0162	0.0178	0.0195	0.0211	0.0227	0.0243
12	0.0127	0.0146	0.0164	0.0187	0.0210	0.0234	0.0255	0.0273	0.0292
14	0.0148	0.0169	0.0191	0.0213	0.0236	0.0263	0.0290	0.0318	0.0341
16	0.0154	0.0193	0.0218	0.0242	0.0267	0.0292	0.0322	0.0352	0.0383
H (FT)	DEFLECTION COEFFICIENTS								
6	0.0020	0.0030	0.0042	0.0058	0.0077	0.0100	0.0127	0.0159	0.0196
8	0.0026	0.0040	0.0056	0.0077	0.0103	0.0133	0.0170	0.0212	0.0261
10	0.0032	0.0049	0.0070	0.0097	0.0129	0.0167	0.0212	0.0265	0.0326
12	0.0036	0.0057	0.0084	0.0115	0.0154	0.0200	0.0255	0.0318	0.0391
14	0.0041	0.0063	0.0093	0.0132	0.0179	0.0233	0.0296	0.0371	0.0456
16	0.0042	0.0069	0.0102	0.0144	0.0198	0.0265	0.0338	0.0423	0.0521

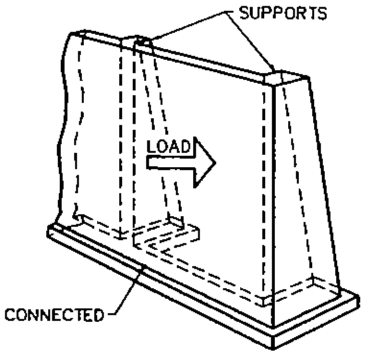


CASE 1: 2-SPAN WALL CONTINUOUS

DESIGN MODEL

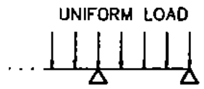


MOMENT (OVER SUPPORT) = -1.00
MOMENT (MID SPAN) = 0.50
DEFLECTION (BETWEEN SUPPORTS) = 0.40

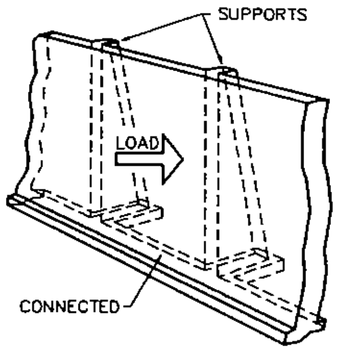


CASE 2: END SPAN OF 3 OR MORE SPANS

DESIGN MODEL



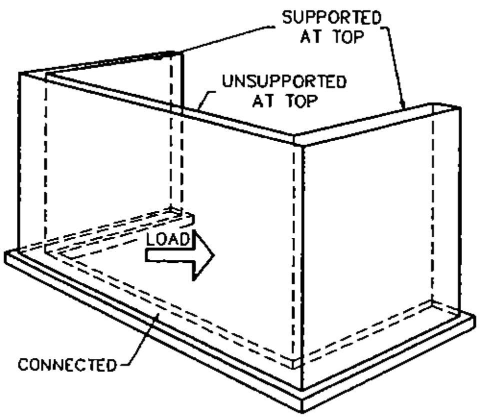
MOMENT (OVER SUPPORT) = -0.84
MOMENT (BETWEEN SUPPORTS) = 0.60
DEFLECTION (BETWEEN SUPPORTS) = 0.52



CASE 3: INTERIOR SPAN OF 3 OR MORE SPANS

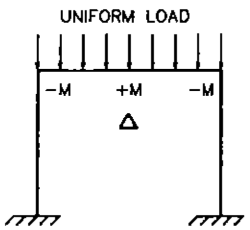
DESIGN MODEL

MOMENT (OVER SUPPORTS) = -0.68
MOMENT (MID SPAN) = 0.35
DEFLECTION (MID SPAN) = 0.24



CASE 4: WALL CONTINUOUS WITH SUPPORTED CROSS WALLS

DESIGN MODEL



$-M = -0.48$
$+M = 0.52$
$\Delta = 0.43$

$$\frac{82080}{E_c\ I_c\ (actual)}$$

Example 7: A continuous sound deflection wall similar to that shown in Fig. 6, case 2 and 3 is 10 ft high and spans 18 ft between buttresses. (a) Determine the factored moment, M_u, and the deflection for a 23 psf wind load when standard Insteel 3-D panels made with $f'_c = 2500$ are used (b) Determine the deflection if the minimum concrete strength, $f'_c = 4000$ psi.

From Table 1: Moment coefficient = 0.0211
Deflection coefficient = 0.0163

a. For end span (Case 2, Fig. 6):
Max. $M_u = 0.0211\ (23)\ (0.84)\ (1.3) = 0.53$ ft-kips/ft
$\Delta = 0.0163\ (23)\ (0.52) = 0.195$ in.
For interior span (Case 3)
Max. $M_u = 0.0211\ (23)\ (0.68)\ (1.3) = 0.43$ ft-kips/ft
$\Delta = 0.0163\ (23)\ (0.24) = 0.090$ in.

Note: From page 6, the resisting moment, $\phi\ M_n$ is 1.30 ft-kips/ft

b. *For* $f'_c = 4000$ *psi*, $E_c = 57\sqrt{f'_c} = 3605$
$E_c\ I_c(actual) = 3605\ (28.8) = 103{,}824$

$$End\ span\ \Delta = \frac{82{,}080}{103{,}824}\ (0.195) = 0.79\ (0.195) = 0.154\ in.$$

$$Interior\ span\ \Delta = 0.79\ (0.090) = 0.07\ in.$$

Example 8: The gabled end of a house has ICS 3-D panels to the eave line, 8′0″. The gable above the panels is constructed of wood framing that cannot provide support to the top of the wall. The end dimension is 30 ft. Determine the maximum moment and maximum deflection of the 3-D wall under a hurricane wind load of 27 psf.

From Table 1: Moment coefficient = 0.0288
Deflection coefficient = 0.0606

Max. $M_u = 0.0288\ (27)\ (0.52)\ (1.3) = 0.526$ ft-kips
$\Delta = 0.0606\ (27)\ (0.43) = 0.70$ in.

$$\frac{0.70}{30 \times 12} = \frac{1}{514}$$

Example 9: Same as example 7 except use Table 2.

a. For end span (Case 2)

Max. $M_u = 0.0230\ (23) = 0.53$ ft-kips/ft
$\Delta = .0085\ (23) = 0.195$ in.

b. For interior span (Case 3)

Max. $M_u = 0.0186\ (23) = 0.43$ ft-kips/ft
$\Delta = 0.0039\ (23) = 0.09$ in.

Example 10: Same as example 8, except use Table 2, Case 4.

Max $M_u = 0.0195\ (27) = 0.526$ ft-kips
$\Delta = 0.0261\ (27) = 0.70$ in.

B. DERIVATION OF THE TABLES

The coefficients in Tables 1 and 2 are derived by converting stress and deflection formulas given in "Roark's Formulas for Stress and Strain"[1] to units that are convenient to use for the design of ICS 3-D panels. For this derivation the following notation applies:

σ = stress, psi
b = span, in.
y = deflection, in.
M = moment, in-lb
S = section modulus, $in.^3$
q = load, psi
t = thickness, in.
E = modulus of elasticity, psi
I = moment of inertia, $in.^4$
C = arbitrary coefficient

From the above reference, Table 26, Case 2:

$$Max.\ \sigma = \frac{\beta q b^2}{t^2}$$

$$Max.\ y = \frac{\alpha q b^4}{E t^3}$$

a/b	0.50	0.667	1.0	1.5	2.0	4.0
β	0.36	0.45	0.67	0.77	0.79	0.80
α	0.080	0.106	0.140	0.160	0.165	0.167

[1]Young, Warren C. "Roark's Formulas for Stress and Strain," Sixth Edition, McGraw-Hill Book Company, New York, 1989.

$$\sigma = \frac{M}{S} = \frac{\beta q b^2}{t^2}$$

For 1 in. width

$$S = \frac{t^2}{6}$$

$$\frac{M(6)}{t^2} = \frac{\beta q b^2}{t^2}$$

$$M = \frac{\beta q b^2}{6}$$

Therefore, in Table 1:

$$\textit{Moment coefficient} = \frac{\beta b^2}{6} \quad b = \textit{span between supports.}$$

Values of β are determined by straight-line interpolation from the Roark table above.

Example: H = a = 10 ft
b = 20 ft   a/b = 0.5

$$\textit{Moment coefficient} = \frac{0.36(20)^2/6}{1000} = 0.024$$

(Note: The division, 1000, is necessary when entering with psf and yielding ksf.)

$$y = \frac{\alpha q b^4}{Et^3} = \frac{Cqb^4}{EI} \textit{ where C depends on support conditions.}$$

For 1 in. width, $I = t^3/12$

$$y = \frac{Cqb^4(12)}{Et^3} = \frac{\alpha qb^4}{Et^3} \qquad C = \frac{\alpha}{12}$$

$$y = \frac{\alpha qb^4}{12EI} \textit{ in consistant units}$$

For $q = lb/ft,\ b = ft,\ E = psi,\ I = in.^4$

$$y = \frac{\alpha qb^4\ (144)}{EI}$$

Example: H = a = 16 ft
b = 16 ft a/b = 1.0

For standard 3-D Panel $f'_c = 2500$ psi

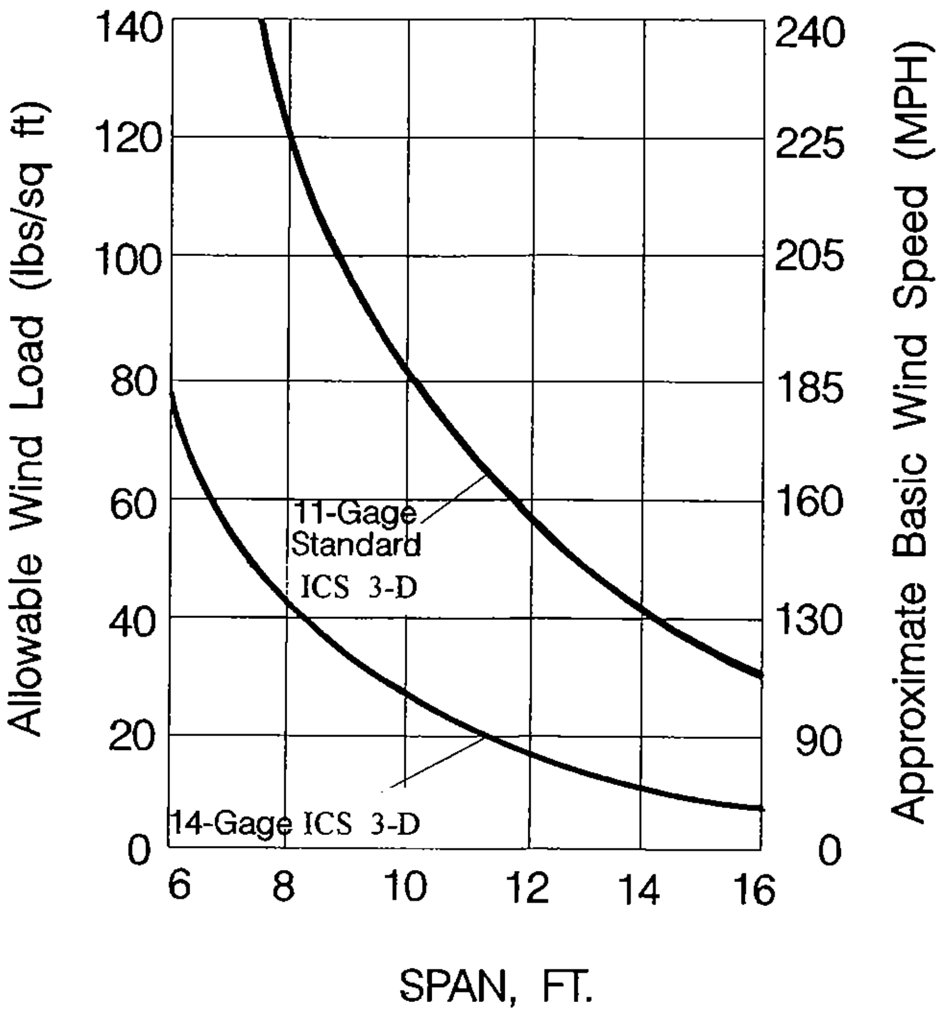
EI = 82,080 (See previous section)

q = 1 psf = 0.001 ksf

$$y = \frac{0.140(0.001)(16)^4(144)}{(82,080)} = 0.0161$$

Since the coefficients given in the reference are based on simple supports on three sides, adjustments have been made to include the effects of continuity. The coefficients in Fig. 6 are derived by comparing the moments and deflections of one-way continuous beams (Cases 1, 2 and 3) or frames (Case 4) with simple span beams. Thus, by multiplying the coefficients derived above by the coefficients shown in Fig. 6, reasonably close approximations can be made.

3-D Wind Load Capacity



*Load values in accordance with ASCE 7-88 (ANSI A58.1) for category I structures, Exposure C, Elevation 0-15'.

3-D PANEL OFFERS DESIGN FLEXIBILITY

The 3-D system allows the architect/engineer to design for various wind loads and seismic conditions. By varying the size of the wire reinforcement, thickness of shotcrete and incorporating formed-in place beams and columns, most structural requirements can be met. The 3-D system can be used in many building applications; residential, commercial, industrial and institutional; and is designed to meet standard building code requirements. The 3-D system can be used in single-story or multi-story facilities, curtain walls, privacy walls, sound barrier walls, load bearing walls, non-load bearing walls, retaining walls and roofs. ICS 3-D Panels Works, Inc. is available to assist you with design and estimating details.

CALCULATIONS