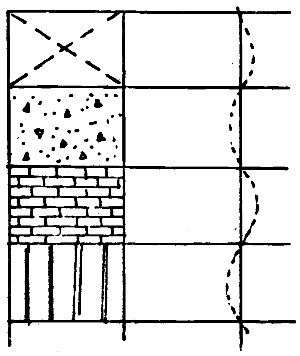
**BRACING REQUIREMENTS**

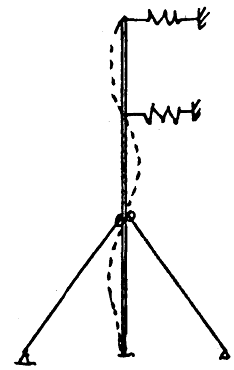
Brace requires STIFFNESS & STRENGTH

- Depends on the type of brace i.e. whether the brace controls the movement at a single particular point or whether it controls the relative movement between two points (or stories)

RELATIVE



SINGLE POINT



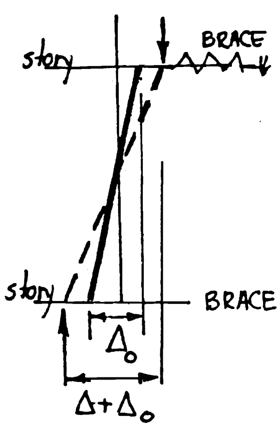
THEORY:

STIFFNESS, $\beta = \frac{P}{L}\left(1 + \frac{\Delta_o}{\Delta}\right)$ $\qquad$ $\beta = A\frac{P}{L}\left(1 + \frac{\Delta_o}{\Delta}\right)$ where A varies between 2.0 and 4.0

STRENGTH, $F_{br} = \frac{P}{L}(\Delta + \Delta_o)$ $\qquad$ $F_{br} = 2\frac{P}{L}(\Delta + \Delta_o)$

└ horizontal component

where P = Sum of column loads in a story to be stabilized by the brace. In the case of a point brace P would be the average load in the compression member above and below the brace point. In the case of a beam or beam-column P would be the compressive force in the member.




L = Story height or distance between braces

$\Delta_o$ = small displacement from straight position, at the brace point caused by sources other than the gravity loads or compressive forces. For example $\Delta_o$ would be a displacement caused by wind or other lateral forces, erection tolerance (initial out-of-plumb), etc.

$\Delta$ = additional displacement at the brace point as a result of the compressive forces or gravity loads.

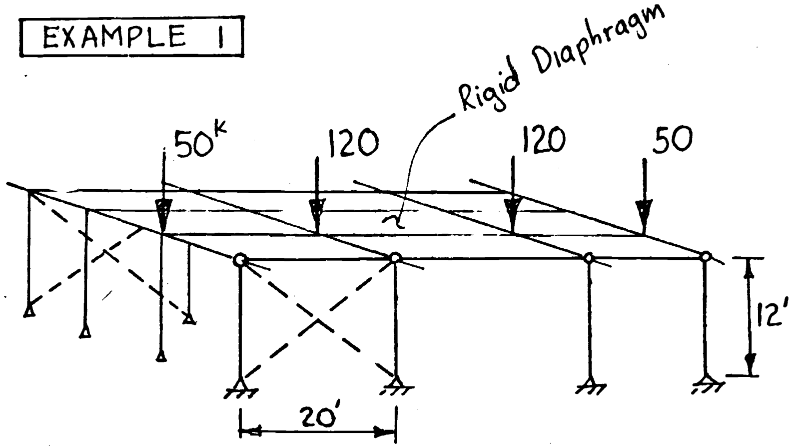
NOTE: TOTAL SMALL DISPLACEMENT AT THE BRACE POINT $= \Delta + \Delta_o$
$\Delta$ and $\Delta_o$ are measured relative to adjacent brace points.

DESIGN: Assuming $\Delta = \Delta_o$ and $\Delta_o = 0.002L$ and using a safety factor of 2.0 in stiffness requirement (factor of safety for strength is handled directly by allowable stress) the theory requirements above become: conservatively uses A = 4.0

DESIGN FORMULAS

$\boxed{\beta_{REQ'D} = 4P/L \;;\; F_{br} = 0.004P}$ RELATIVE $\qquad$ or $\qquad$ $\boxed{\beta_{REQ'D} = 16P/L \;;\; F_{br} = 0.008P}$ SINGLE POINT

## EXAMPLE 1




REQ'D: SIZE OF TENSION BRACES TO STABILIZE THE SYSTEM

$P/\text{brace} = 3(50+120+120+50) = 1020\text{k}$

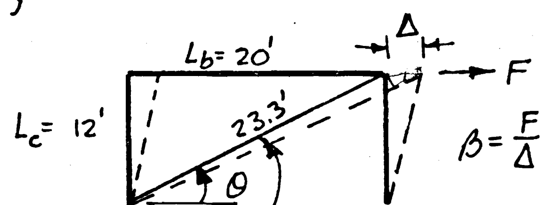
RELATIVE BRACE

$$\beta_{REQ'D} = 4P/L = 4(1020)/12' = 340^{k}/_{FT}$$

$$F_{br} = 0.004(1020) = 4.08\text{ k}$$

These requirements assume the brace is perpendicular to the columns to be braced

Tension Bracing system every third bent
Roof acts as rigid diaphragm to support "unbraced" bents
$F_y = 36$ ksi (See Ex. 4 for a case with gravity plus wind loads)




$A_{br}$ = area of brace
$L_{br} = 23.3'$

Brace Force $= F_{br}/\cos\theta = 4.08/\cos\theta = 4.76^k$

$$F_t = 0.6F_y = 22\text{ ksi} \;;\; A_{br_{NET}} = \frac{4.76}{22} = 0.216\text{ in}^2$$

Assuming rod is threaded - $\underline{\tfrac{5}{8}\phi}$ REQ'D

Brace Stiffness

$$\left(\frac{A_{br}E}{L_{br}}\right)\cos^2\theta = \frac{F}{\Delta} = 340^{k}/_{FT}$$

$$\frac{A_{br}(29{,}000)}{23.3}\left(\frac{20}{23.3}\right)^2 = 340 \;;\; A_{b_{gross}} = 0.372\text{ in}^2$$

STIFFNESS GOVERNS ⟶ $\underline{3/4\,\phi}$ REQ'D

$$\boxed{\text{USE } \tfrac{3}{4}\phi \quad F_y = 36\text{ ksi}}$$

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## EXAMPLE 2

- same problem as Ex. 1 but use steel shear diaphragms instead of tension rods - assume corrugated sheet

Brace requirements are the same: $\beta = 340^{k}/_{FT}$, $F_{br} = 4.08$ k

Try 20 ga. ($t = 0.036$ in) -get strength and stiffness of corrugated sheet from American Iron and Steel Institute Booklet "Design of Light Gage Steel Diaphragms)

$\boxed{\text{USE 20ga}}$

Allowable strength $= 0.22^{k}/_{FT}$ $F_{allow} = 0.22 \times 20' = 4.4^k > 4.08^k$ ok

Stiffness ($t = 0.036$ and $L = 20'$) $= 1820^{k}/_{FT} > 340$ req'd ok

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## EXAMPLE 3

-same problem as Ex. 1 but use brick shear wall

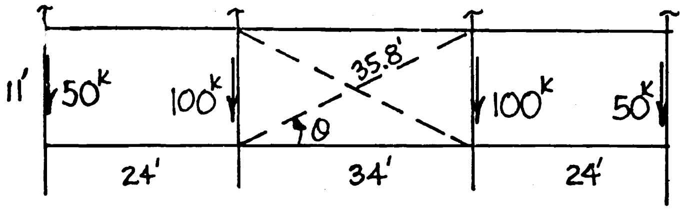
Try 4 in. brick wall - From "Recommended Building Code Requirements for Engineered Brick Masonry" min allow shear stress = 40 psi

$\boxed{\text{USE 4 in brick}}$

$F_{allow} = 40[4 \times 20(12)] = 38.4^k > 4.08^k$ ok

Stiffness - From "The Behavior of One Story Brick Shear Walls" by Benjamin & Williams Proc. ASCE Vol. 84 July 1958 $\beta = 200\,{}^{L_b}/_{L_c} \times$ brick thickness $= 1330^{k}/_{FT} > 340$ ok

## $\boxed{\text{EXAMPLE 4}}$ - Multistory Frame




Type 2 Construction - simple framing, $F_y = 36$ ksi
Bracing every third bent
Wind shear per bent (this level) = $6.5^k$

Bracing req'd to stabilize
Wind shear $= 3(6.5^k) = 19.5^k$
Col. gravity loads $= 3(50+100+100+50) = 900^k$

Brace stability requirements - relative brace

$$F_{br} = 0.004\,(900^k) = 3.6^k$$
$$\beta_{REQ'D} = 4P/L = 4\,(900)/11' = 328\ ^k/FT$$

Notes:
1. Bracing must be adequate to stabilize the structure under gravity load alone and combined wind and gravity load.
2. Axial loads in the columns due to gravity load alone are shown for a typical bent. Axial loads are the same in the "unbraced" bents. Floor diaphragms or bracing are used to transmit the wind shears and PΔ overturning moments to the braced bent.
3. Even though the wind forces alter the distribution of the axial forces in the columns, the sum of the column loads must be equal to the applied gravity loads so the shears due to the PΔ moments are unaffected.
4. Use 33% increase in allowable stresses for the combined load case.

<u>Gravity</u>:

Stiffness: $\dfrac{A_{br}E}{L_{br}}\cos^2\theta = 328\ ^k/FT$ ; $A_{br\,gross} = \dfrac{328\,(35.8)}{29{,}000}\left(\dfrac{35.8}{34}\right)^2 = 0.451\ in^2$

Strength: $F_{br} = 0.004\,P = 0.004(900) = 3.6^k$  $F_b/\cos\theta = 3.8^k$

$$A_{br\,net} = 3.8/.6F_y = 3.8/22 = 0.173\ in^2$$

<u>Gravity plus Wind</u>:

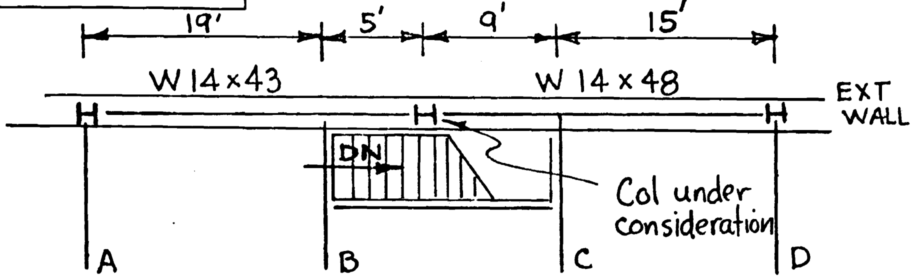
Stiffness: - no change from gravity  $A_{br\,gross} = 0.451\ in^2$

Strength: $A_{br} = \dfrac{19.5}{1.33(22)\cos\theta} + \dfrac{3.8}{1.33(22)} = 0.830\ in^3$ ← controls
(Anet)

$\boxed{\text{use } 1\tfrac{1}{4}\phi \text{ Threaded Rod } (A_{Net} = 0.969\ in^2)}$

Note that the brace strength requirement for gravity loads is added to the requirement for wind alone. Do <u>not</u> add the larger stiffness requirement.

## EXAMPLE 5 - POINT BRACES




12'-0" Story height

Column load above = $175^k$
Column load below = $200^k$

Exterior column in a multistory frame. The only lateral support of the column under consideration at the floor levels is provided by the weak-axis bending strength and stiffness of the W sections shown. Headroom requirements do not permit a beam to frame directly into the web of the column. Determine if these beams have sufficient strength and stiffness to brace the column at the floor level. It is assumed that the spandrel beams are laterally braced at locations A, B, C and D.

The beams act only to control the column movement at this particular floor level so they are single point braces. Therefore the bracing requirements are:

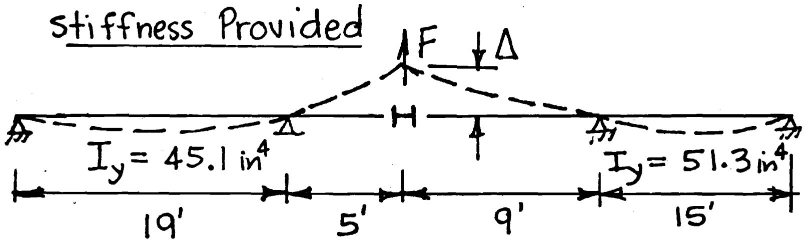
$$\beta_{REQ'D} = 16P/L \quad \text{and} \quad F_{br} = 0.008P$$

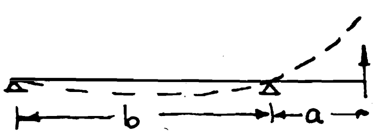
$P$ = avg. column load = $(175+200)/2 = 188^k$ ; $L = 12'$

$$\beta_{REQ'D} = \frac{16(188)}{12} = 250^{k/'}$$

$$F_{br} = 0.008(188) = 1.50^k$$

Stiffness Provided




$F_1$ $\Delta_1 = \dfrac{F_1 a^2(a+b)}{3EI}$ (AISC Manual) $\qquad \beta = \dfrac{F_1}{\Delta_1} = \dfrac{3EI}{a^2(a+b)}$

W 14×43 ; $\beta_1 = 3(29000)\,45.1/(5)^2(24)\,144 = 45.4$ k/FT
W 14×48 : $\beta_2 = 3(29000)\,51.3/(9)^2(24)\,144 = 28.7$ k/FT

$\beta_{TOTAL} = 74.1 < 250$ k/FT N.G.

~~TRY~~ USE

W 14×78 left side $\quad \beta = 208$ k/FT
W 14×61 right side $\quad \beta = 60$

$> \beta_{TOTAL} = 268 > 250$ k/FT OK

Check strength - The $1.50^k$ required force is provided by the beams in proportion to their stiffnesses

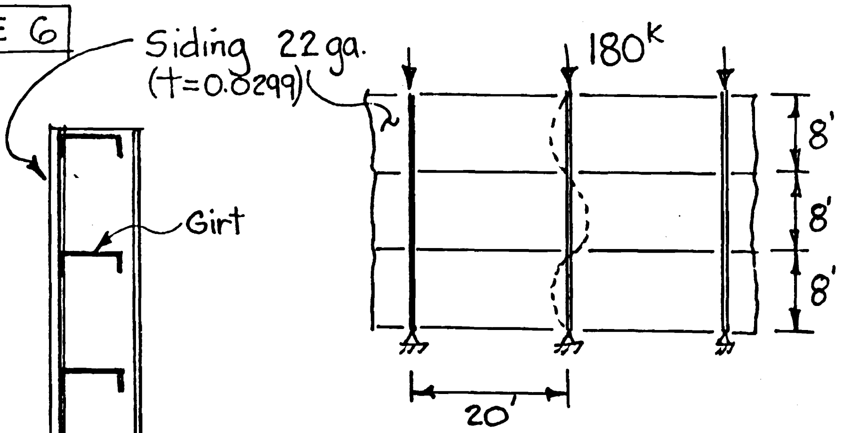
W 14×78 ($S_y = 34.5\,in^3$)

$F_{W14x78} = 1.50\left(\frac{208}{268}\right) = 1.17^k$ ; $M_y = 1.17^k(5\times12) = 70$ in-k

$f_{by} = M_y/S_y = 70/34.5 = 2.0$ ksi OK when considered with $f_{bx}$ also

$\boxed{\text{USE W 14x78 and W 14x61}}$ - COL CAN NOW BE SAFELY DESIGNED FOR KL = 12'

EXAMPLE 6




Is the 22ga. siding sufficient to support the girts so that the columns are braced at the one-third points? Siding is attached to the girts in a standard manner using screws for fasteners.

Assume a 20 ft width of siding supports each column
From AISI Booklet "Design of Light Gage Steel Diaphragms"
22 ga × 20'

$F_{allow} = 0.487\ ^{k}/_{FT} \times 20' = 9.74^{k}$

$\beta = 105\ ^{k}/_{FT}$

Relative Brace (siding prevents the girts from moving relatively)

$F_{REQ'D} = 0.004(180) = 0.72^{k} < 9.74$ OK

$\beta_{REQ'D} = 4P/L = 4(180)/8 = 90 < 105\ ^{k}/_{FT}$ OK

$\boxed{\text{22ga. Siding OK}}$

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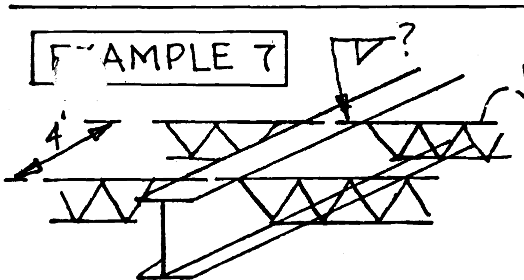
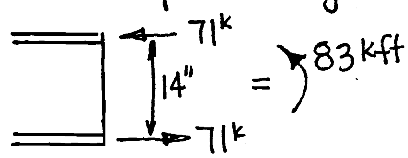
EXAMPLE 7




M = 83 kft

How much weld is required so that the joist will adequately brace the beam

Treat the compression region of the beam as a column.




$P = 71^{k}$

$\beta_{REQ'D} = 71\ ^{k}/_{FT}$

Typical metal floor decks provide approximately 10 times this required stiffness. Metal deck with 2½ in. concrete fill provides 30 times this stiffness. Normal fasteners that connect the deck to the joists can transfer the 0.57k force.

Point brace (for connector requirements)

$F_{br} = 0.008\,P = 0.008(71) = 0.57^{k}$

USE $\frac{3}{16}$ 0-$\frac{1}{4}$" (Tack)

Floor system - diaphragm - relative brace prevents the relative movement of adjacent joists

$\beta_{REQ'D} = 4P/L = 4(71)/4' = 71\ ^{k}/_{FT}$

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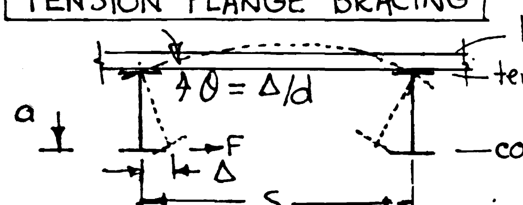
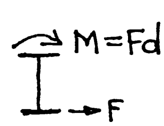
TENSION FLANGE BRACING




brace - must have bending stiffness $I_b$




$\sqrt{}\ \beta_{REQ'D} = 16P/L$ ; $\sqrt{}\ F_{br} = 0.008\,P$

$\beta_{act.} = \frac{F}{\Delta} = \frac{4EI_b}{sd^2}$ - solve for $I_b$

assumes no cross section distortion; check brace for bending moment, $M/2 = Fd/2$