Folded Steel Plate Roof for a Suburban Branch Bank

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THE ARCHITECTURAL design of the Union National Branch Bank building, Youngstown, Ohio, was keyed to the need for a structure with a striking appearance to serve as the focal point in a suburban shopping center.

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To achieve this appearance, architect P. Arthur D'Orazio called for a circular building with a folded-plate roof and glass exterior walls.

The author prepared preliminary structural studies for the project. These studies pointed to steel as the structural material that would fill the architectural requirements to the best advantage.

Photo by A. Pavliga

Fig. 1. Union National Branch Bank, Youngstown, Ohio

CONSTRUCTION DETAILS

The building is 50 ft-0 in. in diameter. The exterior walls consist of glass between the columns, from a curb at the floor to the roof plates (Fig. 1). The roof is made of folded steel plates, and is divided into 28 segments radiating from a skylight at the center (Fig. 3). Each segment is made in the form of an inverted "V".

The roof is supported around the outside edge of the circle by 28 columns, one under each segment. Tubular columns were chosen to facilitate window installation and for a clean architectural effect. Each tubular column is provided with a drain (see Figs. 2 and 3). A water stop is welded at each column (Sect. F-F of Fig. 3) to provide a dam to direct water into the individual column drains. Water accumulating on the cantilever section of the roof beyond the stiffener plate is permitted to drip from the valley. A small facia angle was also provided along the outmost edge of the plate.

At the center of the 50 ft-0 in. circle, a 7 ft-3 in. diameter skylight is provided. The skylight is placed in a steel plate cylinder with angle stiffeners or rings on the inside of the cylinder to provide adequate strength. A vault at the rear of the building interrupts the glass walls. Short columns in this area bear on the vault roof.

DESIGN PROCEDURE

The problem was approached in the following way:

1. Treat each segment as a beam cantilevered from the central skylight ring. Determine dead and live loads, column reaction. Compute moment of inertia. Compute and draw shear and moment diagrams and determine stresses in the plate. Check thickness of plate elements for compliance with code requirements.
2. Check plate in a transverse direction for bending.
3. Check column size used for direct load and wind.
4. Check bent for wind and twisting.
5. Check compression tension ring.

The design computations, in a somewhat abbreviated form, are shown in Fig. 4.

The design was based on the 1949 AISC Specification, using A7 steel.

FABRICATION AND ERECTION

The bending of the lip along the lower edge of the plates was done in two stages because the length of the plate was too long for the press brake available. A cut was made at the midpoint of the lip to permit two separate bending operations of the press brake. This cut was welded after bending was complete. This bend along the lower plate edge had a varying angle from the outer edge of the roof to the ring. Temporary stiffeners or ties were provided to hold the shape of the plates until after erection and field welding was complete.

The roof, as first conceived, was to receive one of the newer types of plastic roofing material with insulation applied on the underside of the roof. Near the time of bidding, it was discovered that the manufacturer would not guarantee results in this case. For this reason, the usual built-up roof was specified with a sprayed plaster ceiling.

Erection of the structure was accomplished by building a scaffold at the center of the circle beneath the skylight ring. Columns were then erected and the folded plates were placed on the columns and seated on the angle provided on the ring for this purpose. Welding of column caps and at the compression tension ring was completed before the scaffold was lowered. Some small areas inaccessible for welding because of the scaffold were welded after the scaffold was removed. A valley dam and column stiffener at the columns were also welded at this time.

CONCLUSION

This paper is not intended to be the last word on the design of a folded plate roof of this type. It is one engineer's approach and treatment of this problem. There is never enough time to treat a new problem or situation as we would like. With more time, this structure could possibly have been refined and the cost reduced. There are, no doubt, other approaches that could have been made.

Fig. 2. Plate dam beyond each drain helps to tunnel water into drain and stiffen the folded plate over the column

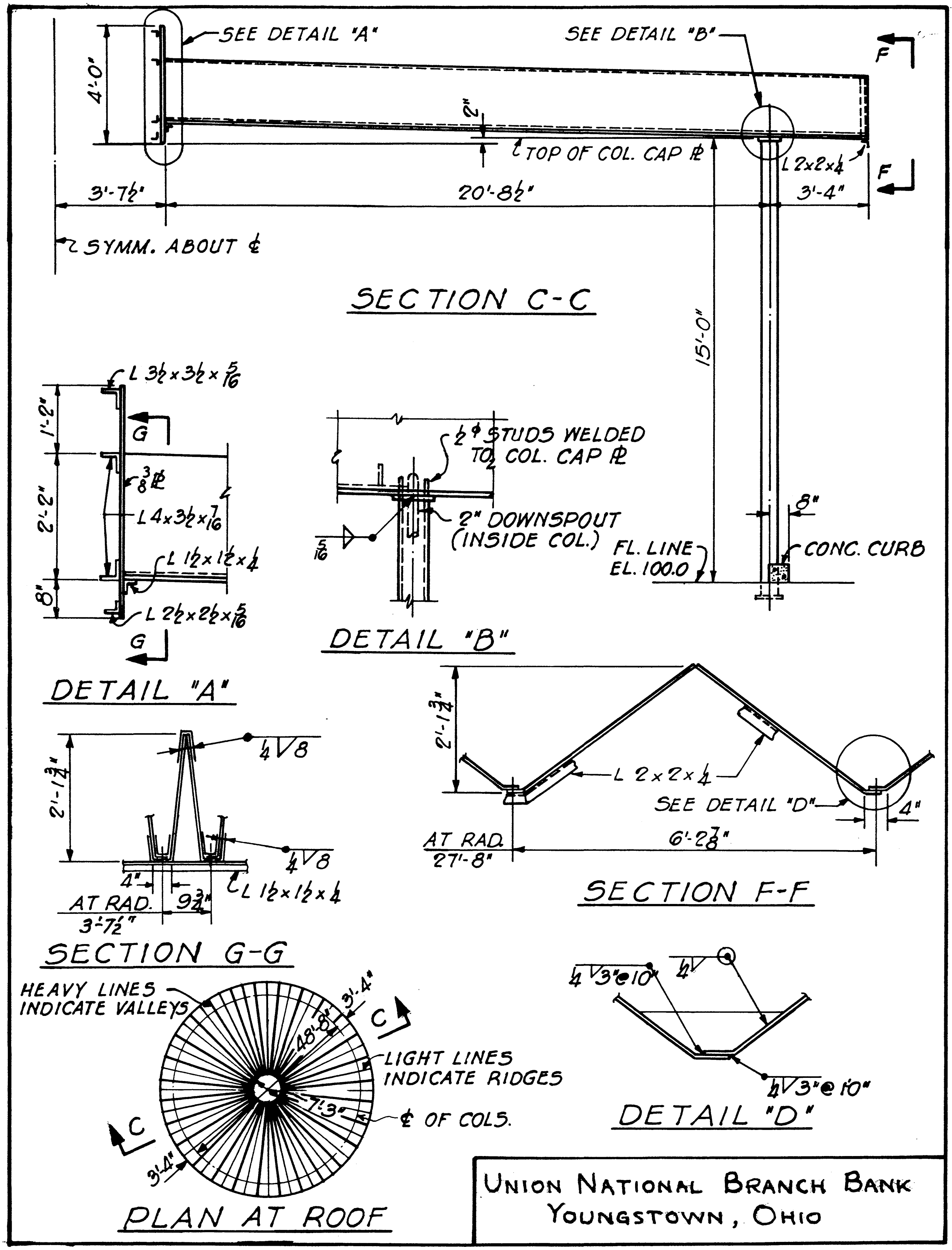
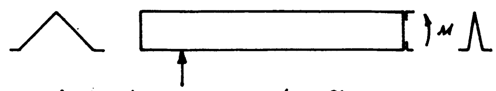


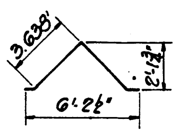
Fig. 3. Details of roof framing

CALCULATIONS

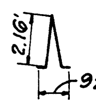
1. Treat each segment as a beam between column and compression-tension ring:



Compute loads: L.L. = 30 lbs/sq. ft.
$\frac{1}{4}$" steel plate = 10.5 lbs/sq. ft.
Built-up roof & insulation = 6.0 lbs/sq. ft.
Sprayed-on plaster ceiling = 8.0 lbs/sq. ft.
Skylight & framing = 800 lbs (total)
Est. total wt. of $\frac{1}{4}$" plate = 44,500 lbs
Wt. of compression-tension ring = 2,000 lbs

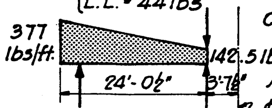


(2)(3.638 + 0.25)(24.5) = 190.51
(30)(6.206) = 186.18
376.69 lbs/1st ft.



(2)(2.16 + 0.25)(24.5) = 118.09
(30)(0.813) = 24.39
142.48 lbs/last ft.

D.L. = 2,800/28 = 100 lbs
L.L. = 44 lbs

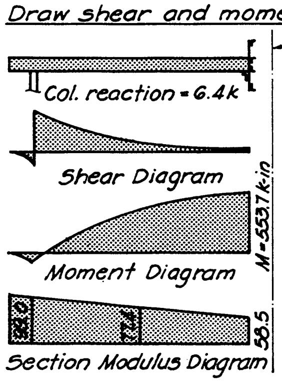


LOADING DIAGRAM
(Assume load varies uniformly between edge and ring.)

Col. reaction $= \frac{377+142.5}{2}(24.04) + 144$
$= 6,388$ lbs (Use 6.4 k)

Mom. at ring $= (64)(20.7) - (0.1425)(24.04)^2(\frac{1}{2})$
$- (0.2345)(24.04)^2(\frac{1}{2})(\frac{1}{3})$
$= 132.5 - 41.2 - 45.2$
$= 46.1$ k-ft.
$= 553.7$ k-in

Draw shear and moment diagrams:



Note from diagrams that section modulus increases from column to ring while bending moment decreases.

Check folded plate:

$S_{req'd.} = \frac{554}{20} = 27.7 < 58.5$ in.^{3} OK

Treat plate as a beam or girder web & check thickness:

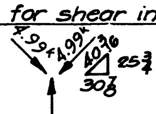
$\frac{\text{℞ thickness}}{\text{℞ depth (sloped)}} = \frac{0.25}{40\frac{3}{16}} = \frac{1}{160.7} > \frac{1}{170}$ (Spec. Sect. 26b)

Max. allowable shear with no stiffeners (Spec. Sect. 26e)

$h/t = 160.7$

$V = \frac{64 \times 10^6}{(h/t)^2} = \frac{64,000,000}{(160.7)^2} = 2,480$ psi (allowable)

Check for shear in plate at column:



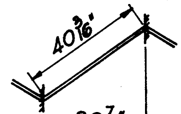
$\frac{3.2}{25\frac{3}{4}} = \frac{P}{40\frac{3}{16}}$

$P = 4.99^k$

$V = \frac{4,990}{(0.25)(40\frac{3}{16})} = \frac{4,990}{10.04} = 497$ psi

∴ No stiffeners req'd.

2. Check plate for bending (lateral) only on surfaces up to a 45 degree slope.
(Consider ends fixed because of lateral continuity.)



Section at Col.

Load $= 30 \times 2'$-$6\frac{3}{8}" = 77.0$ lbs (on horiz. area)
$= 24.5 \times 3'$-$4\frac{3}{16}" = 81.6$ lbs (on slope)
159 lbs

Mom. (ends) $= \frac{Wl}{12} = \frac{(159)(40\frac{3}{16})}{12} = 530$ lb-in

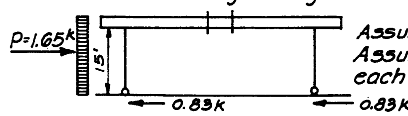
$S_{\frac{1}{4}\text{"℞}} = \frac{bd^2}{6} = \frac{(12)(0.25)^2}{6} = 0.125$ in.^{3}

$f = \frac{M}{S} = \frac{530}{0.125} = 4,240$ psi (max. lateral bending at cols.) OK

3. Check columns:
6x4 rectangular tube with $\frac{5}{16}$" wall
Area = 5.52 sq. in.
$S_y = 6.73$ $r_y = 1.56$
$S_x = 8.45$ $r_x = 2.14$

$\frac{L}{r} = \frac{(15')(12)}{1.56} = 115.5 < 120$ OK (Spec. Sect. 15a(2))

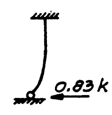
4. Check for wind:
Consider one bay or segment.



Assume pin end col. bases.
Assume $\frac{1}{2}$ wind load on each col. base.

Mom. at top of leeward col. = (0.83)(15) = 12.45 k-ft.

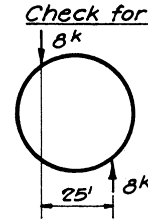
Check for combined stress:



$\frac{P}{A} = \frac{6.29}{5.52} = 1.14$ ksi $\quad \frac{1.14}{(10.53)(1.33)} = 0.081$

$\frac{M}{S} = \frac{(124.5)(12)}{8.45} = 17.67$ ksi $\quad \frac{17.67}{(20)(1.33)} = \frac{0.663}{0.744} < 1.0$ OK

Check for wind (lateral twisting)



Wind force = (25)(16)(20) = 8.0 k
Moment = (8)(25) = 200 k-ft.
Force at top of each col.

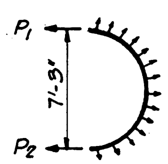
$= \frac{200}{(28)(25)} = 0.286$ k

Col. mom. = (0.286)(15) = 4.28 k-ft. = 51.4 k-in.

$\frac{P}{A} = \frac{6.4}{5.52} = 1.16 \quad \frac{1.16}{(10.53)(1.33)} = 0.083$

$\frac{M}{S} = \frac{51.4}{6.73} = 7.64 \quad \frac{7.64}{(20)(1.33)} = \frac{0.286}{0.369} < 1.0$ OK

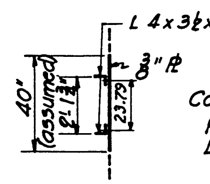
5. Check section modulus of ring:



Mom./ft. $= \frac{(553.7)(12)}{9\frac{3}{4}} = 681$ k-in.

Mom. at ℄ ring $= \frac{(7.25)(681)}{2} = 2,470$ k-in.

$S_{req'd.} = \frac{2,470}{20} = 123.5$ in^3



Compute Moment of Inertia:
Plate = 2,000
Ls = 2(3.09)(11.85)^2 = 873
2,873 in^4

Section modulus $= \frac{2,873}{20} = 143.7 > 125.5$ in.^{3} OK

Fig. 4. Design calculations